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UNIVERSITY OF ALBERTA

DESIGN OF HYDRAULIC FILL

BY

© ANGELA M. AMARAL GURGEL KÜPPER

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE
OF DOCTOR OF PHILOSOPHY

DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA
SPRING 1991

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to

João, Arthur and Suzana

Abstract

Hydraulic fill is comparable to other construction materials in the sense that both the composition of the mix and the placement method affect the properties of the material and therefore must be designed so the the fill performs satisfactorily. However, it is necessary to understand the factors that influence the behaviour of hydraulic fills in order to design it properly.

An experimental approach was adopted including laboratory flume deposition tests and large scale field deposition tests. Three different sands were used for the flume tests which studied the effect of slurry concentration, flow rate and mean grain size on the properties of the fill such as: geometry, density, grain size distribution and fabric. Eight large scale field tests were carried out on a tailings dam to study the deposition process and the effect of the placement method on the fill characteristics (geometry, density, grain size distribution, fabric and fines capture). An instability of the deposition process was observed in the field and its potential causes and consequences were discussed, showing the importance of having relatively constant feed parameters. A comparison between flume tests and field tests results proved that flume tests are a valuable tool in the study of hydraulic fills.

The results of the flume and field tests showed a consistent trend of fill slopes becoming steeper as the discharge flow rate increased and as the slurry concentration and mean grain size increased. These conclusions are consistent with observations of other hydraulic fills and natural alluvial deposits. An empirical method of estimating beach slopes based on the discharge parameters is proposed.

The trend of variation of density with the discharge parameters is discussed and the mean density values for the field tests are compared with the steady state line for the material deposited. A sedimentologic approach to the analysis of density of hydraulic fills is discussed. It is suggested that a boundary between the hydraulics of the flow and the geotechnical behaviour of the fill might be on the study of bedforms as they relate the flow energy to the formation of different deposits.

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LIST OF SYMBOLS

$\%F_b$	Amount of fines in the beach material
$\%F_f$	Amount of fines in the feed samples
$\%F_{sp}$	Amount of fines in the spigot samples
A	Cross-sectional area of the discharge pipe
a	regressive constant (Equation 7.3, p.269)
b	regressive constant (Equation 7.3, p.269)
C	Slurry concentration as defined in Equation 3.3, p.59
C_A	Consistency A as defined in Equation 3.4, p.59
C_B	Consistency B as defined in Equation 3.5, p.59
C_U	Coefficient of uniformity (D_{60}/D_{10})
C_v	Slurry concentration in terms of volume (Equation 3.2, p.58)
C_w	Slurry concentration in terms of weight (Equation 3.1, p.58)
$C_{w(sand)}$	Slurry concentration in terms of weight considering only the sand fraction of the solids (weight of sand divided by total weight of the slurry)
$C_{w/f}$	Slurry concentration in terms of weight of the feed samples
$C_{w/sp}$	Slurry concentration in terms of weight of the spigot samples
d	Mean depth of flow
D_n	Diameter at which n percent of the grains are finer
e	Void ratio
f_0	Darcy-Weisbach coefficient
FC	Fines capture efficiency (Equation 5.1, p.150)
g	Acceleration of gravity

H	Maximum beach elevation (Equation 7.2, p.269)
h	Thickness of the layer for which FS = 1 (Equation 7.12, p.276)
h_1	Height of sedimented solids in a container of parallel walls (p.59)
h_2	Height of water above the sedimented material in a container of parallel walls (p.59)
H_f	Height between the spigots and the beach surface at the end of the test
H_i	Height between the spigots and the beach surface at the start of the test
i	Slope
i_{av}	Average slope
i_{ov}	Overall slope
n	a parameter of the normalized beach profile equation (Equation 7.2, p.269)
P'	Slope parameter (Equation 7.17, p279)
q	Specific flow rate (flow rate divided by flow width)
Q_f	Flow rate of fluid
Q_s	Flow rate of sediment
$Q_{t(v)}$	Total flow rate expressed in terms of volume
Q_t,Q	Total flow rate
r_h	hydraulic radius
V	Mean velocity of flow
V_s	Volume of solids in the slurry
V_t	Total volume of slurry
w	Width
W_s	Weight of solids in the slurry
W_t	Total weight of slurry
W_w	Weight of water in the slurry

δ	Thickness of the material having shear strength τ_0
ϕ	Angle of repose
γ	Unit weight
γ_f	Unit weight of the fluid
γs	Specific weight of the grains
γ_t	Unit weight of the deposited material
γ_w	Specific weight of water
μ	Viscosity of fluid
ρ	Density of fluid
ρ_d	Dry density of the beach material
ρ_{max}	"Maximum" dry density of the beach material
ρ_{min}	"Minimum" dry density of the beach material
ρ_s	Density of sediment
τ_0	Shear strength of the material just after settling

Chapter 1

Introduction

Hydraulic fills are fills built by discharging slurry on to an area in such a way that most of the fluid is drained away and most of the solids are deposited to form the fill.

Hydraulic fill has a wide range of applications, as for example: construction of water retention dams, construction of tailings dams, disposal of industrial and mineral wastes, construction of artificial islands for offshore oil exploration and other uses, backfill of underground mines, disposal of dredgings, closure of rivers and seas, reclamation of lowland areas, etc.

Hydraulically deposited materials also occur in nature and cover vast portions of our planet. Hydraulic deposition takes place on river beds and floodplains, on beaches, on the bottom of the ocean, on alluvial fans and along the path of debris flows, to give some examples. There are no essential differences between man-made and natural hydraulic fills. A comparison shows that the physical phenomena are exactly the same, the difference being mainly the larger variability of materials and boundary conditions usually associated with natural deposition.

Hydraulic deposition can be carried out below water (subaqueous deposition) or above the water level (subaerial deposition) forming a beach-like deposit. The physical phenomena associated with subaerial and subaqueous deposition are different. Subaqueous deposition, for example, involves drag and water entrainment, that are not relevant in subaerial deposition. Subaqueous and subaerial deposition are both present in many applications of hydraulic fills.

The advantages of handling granular materials hydraulically include the possibility of having a high degree of automation associated with low labour and relatively little equipment requirements. Also, hydraulic methods are applicable to a wide range of materials and are usually cost-effective when compared with mechanical methods. They are very convenient for transport and deposition of materials that are already in slurry form such as dredged materials and tailings from mining and industrial operations. Hydraulic handling also permits a certain degree of particle separation. High rates of construction can be achieved at a low cost per volume of fill, although the volume of hydraulic fills tends to be larger than the volume of a compacted fill under similar conditions.

Other situations where hydraulic deposition presents advantages are construction of water retention dams over collapsible foundations, closure of wide valleys with thick alluvial deposits on the bottom and construction of underwater fills. Materials that collapse when saturated such as loess and some residual soils may cause problems if present in the foundation of dams. Hydraulic deposition can be an effective construction method in these cases as the foundation is wetted during construction when the fill is still very flexible and the loads are relatively low. Thick alluvial deposits of relatively high permeability can also be problematic as a foundation for a conventional dam, but can be easily embraced in the design of hydraulic fill dams. Hydraulic deposition is also advantageous for submerged fills. Canadian and Soviet experience has shown that hydraulic deposition is a valuable construction method for offshore structures. In shallow waters, hydraulic fills are advantageous over steel or concrete structures in relation to cost, maintenance, service life, material, labour and rate of construction (Kevorkov, 1968). However, the Canadian experience showed that problems may arise as the water depth increases (Mitchell, 1984).

The association of hydraulic excavation, hydraulic transport and hydraulic deposition, which has been referred to as "hydromechanization", enhances the advantages of handling materials hydraulically. It allows continuous operation and high production rates at relatively low costs. In the USSR, hydromechanization has gained wide use as an effective method of mechanizing mining operations and earthworks in general, including fill construction under severe winter conditions in Siberia (Maslyakov and Rozinoer, 1979; Melent'ev, 1980). In the Netherlands hydromechanization has also proved to be a very effective construction method of large dams for closure of sea arms. During the last 15 years more than ten large dams were built by hydromechanization in the Southwestern part of the Netherlands (de Groot et al., 1988).

Some of the disadvantages of hydraulic handling of granular materials are associated with the use of a relatively large amount of water when availability is restricted or subsequent dewatering of the material at high costs. The main disadvantage of hydraulic deposition is probably the formation of soft or liquefiable deposits. The minimization of this problem is a main concern of hydraulic fill design. The formation of flat slopes may also be a disadvantage of hydraulic deposition, as gentle slopes require large amount of sand and large areas.

Despite its many practical and economical advantages, the use of hydraulic fill encounters some resistance. Nowadays, hydraulic deposition tends to be adopted only in cases for which there is an overwhelming economic and practical advantage over alternative methods, such as cases of disposal of materials that are already in slurry form.

The lack of a rational design method for hydraulic fills that could be used with confidence is one of the causes that hampers the use of hydraulic deposition in engineering projects. This limitation is well exemplified by the case of use of hydraulic deposition for construction of water retention dams in North America, which will be discussed in more detail in Chapter 2. The non-existence of a rational method for design makes it understandable why hydraulic deposition is not utilized more often, especially in North America, where issues of liability and insurance exert such a limiting influence on the engineering profession.

A distinction is made here between the design of the hydraulic fill and the design of the hydraulic fill structure. The design of the hydraulic fill refers to the design of the fill itself as an engineering material. The design of the structure involves many other factors such as foundation, seepage, stability, filters, ancillary structures, etc. In most applications, hydraulic fill is a construction material that can form the whole structure or be only a component of the structure. In almost all cases (the exceptions are some deposits of very fine tailings or dredgings), the hydraulic fill has at least some structural function, supporting loads. As such, hydraulic fill is a construction material that has to be engineered to perform adequately under conditions predicted or imposed by design.

A materials technology approach should be adopted for the design of hydraulic fills in the same way as for other construction materials such as compacted fill, concrete, shotcrete, asphalt, ceramics, etc. All these materials have one important characteristic in common, which is that the properties of the final product depend on two main factors:

- the composition of the mix
- the placement method

The composition of the mix includes the types of ingredients (including additives) and the relative amount of each one. The design of many construction materials such as concrete, asphalt, etc. calls for the determination of the components of the mix and the proportion of each one. For example, the design of concrete should specify the type of cement, size of aggregates, amount of water, type of additives, if any, etc. The exact amount of each ingredient should be determined in order to assure that the final product will have the required properties for the specific project. The design of compacted fills also includes the choice of an adequate soil and the determination of the exact amount of water necessary to achieve the desirable properties for the fill. The amount of water in a compacted fill is usually specified to be within rather narrow limits (± 1 or 2%). The same care with the choice and the proportioning of ingredients is also necessary for other construction materials such as asphalt, shotcrete, ceramics, etc.

Once the composition of the mix is designed, the placement method has to be specified. The placement method can have a tremendous impact on materials properties. For example, if concrete is discharged from the top of a high form to fill a pillar it will segregate, and a section will be formed with coarse aggregate only and no cement. Therefore, an inadequate placement method will cause failure of a pillar that was built with the right mix and the right amount of steel. Compacted fills have to be placed in thin uniform layers, each one compacted by the right number of passes of the designated equipment in order to develop properties adequate to support the design loads. Asphalt has to be laid at the appropriate temperature and rate and compacted the proper way to have satisfactory behaviour; and so on for other construction materials. The placement method has an important effect on the final properties of all materials.

Hydraulic fill does not differ from these other construction materials. Both the composition of the mix and the placement method influence the final product and must be designed so that the fill performs satisfactorily. The solid fraction can be chosen in cases where the material will be taken from a borrow area, however the choice may be limited depending on the local conditions. In the case of hydraulic fill for waste disposal, the solids cannot be specified but the engineer has the option of separating the material for placement in different areas or by different methods. The proportion of fluid to solids also has to be designed. Robinsky (1979) recommends a decrease in the amount of fluid to maximize slope and minimize the environmental impact of polluted fluids and/or soft materials. At the other extreme, the Soviet technology on hydraulic fills calls for a high amount of fluid (low slurry concentration) to maximize density. In many cases, however, the proportion of fluids is not designed, but simply left as the one that comes out of the mill, cyclone or dredge. The use of additives in the mix with the objective of improving the properties of the hydraulic fill is still in its infancy. But as the design of other construction materials such as concrete, asphalt, shotcrete, evolved to use a number of different additives, the same might be expected for hydraulic fills and compacted fills. The placement method of hydraulic fills also has to be engineered. Nowadays the placement method is specified to make construction practical and economical, but it could also be designed with the additional objective of enhancing the properties of the fill that are important for each project. However, there is no adequate understanding of the factors that influence the final behaviour of hydraulic fills to make this type of design approach feasible today.

The main objective of the thesis is to study hydraulic fills as an engineering material. It focuses on the effect of the composition of the mix and the influence of the placement method on the properties of the resulting fill. The approach is very practical but an attempt is made to clarify the physics of hydraulic deposition. Analysis is carried out from a geotechnical point of view, however hydraulics and sedimentology concepts are used to help in understanding the physical phenomena.

Chapter 2 gives a perspective of some of the critical problems associated with hydraulic fills in general and for specific applications. It involves both subaerial and subaqueous deposition. However, due to the need to limit the scope of the thesis, only subaerial hydraulic fills were studied in the subsequent chapters.

Chapter 3 describes laboratory flume tests that were carried out to isolate some of the variables involved in hydraulic deposition and to quantify the effects of these variables. The results of these tests are compared with the results of other published flume tests in Chapter 4.

Similar experimental work was also performed in the field where the scale is much larger and the problem more complex. The field tests are described in Chapter 5 and analyzed and compared with flume results in Chapter 6.

Chapter 7 studies in more detail the geometry of hydraulic fills and proposes an empirical method to predict beach slopes at the design stage.

Chapter 8 discusses the critical problem of fill density. It approaches the question of density from a sedimentologic point of view, looking at the hydraulics of a sediment loaded flow over an erodible bed. This seems to provide a feasible approach that leads to an understanding of the mechanisms involved in the formation of deposits of different structures and densities. It establishes a connection between the hydraulics of the flow and the geotechnical requirement of producing an adequate fill.

The last chapter integrates the different aspects of hydraulic fills that were discussed in the previous chapters and summarizes the main conclusions.

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Chapter 2

Hydraulic Fill Structures - A Perspective*

2.1 - INTRODUCTION

It is revealing to compare the treatment of hydraulic fill structures in Sherard et. al., (1963) with that given by Justin, Hinds and Creager (1945). The former reflects the state-of-the-art in design and construction of engineered fill dams in the 1950's and early 1960's while the latter draws on the period 1930's to early 1940's. The emphasis in both is on North American practice and the difference in treatment of hydraulic fill structures is striking.

There is no systematic discussion hydraulic fill structures in Sherard et. al. (1963). Some failures are included in a table listing unsatisfactory performance of earth dams but the number of hydraulic fill entries is modest. The only reference to modern utilization of hydraulic fill techniques is to the Tuttle Creek Dam which was intended to be a compacted earth fill structure with material to be excavated by draglines from a borrow pit below the water table, then loaded on trucks and subsequently compacted by either tractors or vibratory rollers. This proved impractical because the trucks were bogging down and the sand fill was too wet for conventional compaction by either tractor of vibratory rollers. The project was successfully completed by switching to hydraulic fill techniques.

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Dredged sand was brought to the surface of the dam by pipeline and discharged in such a manner that the silty fines were kept in suspension and ultimately wasted by flow into a discharge pipe through the embankment. This separated the silt from the sand. The water-deposited sand was found to be adequately dense without any further compaction. The travel of a tractor only loosened it so that subsequent tractor work was minimized. More than 6 x 10^6 m^3 of material was dredged at about 13,500 m^3/day. The cost of the dredged sand in place was about one half of the cost with the previous unsatisfactory construction procedure.

The treatment of hydraulic fill structures in Justin, Hinds and Creager (1945) is much more extensive. The stability of hydraulic fill dams is discussed and Gilboy's (1934) classical study on the mechanics of hydraulic fill dams is presented with examples. In addition to this, more than twenty pages are devoted to matters of depositional details and a discussion of some case histories. Clearly in the period between the publication of the two books, hydraulic fill methods fell out of favour and ultimately could no longer be regarded as a common method of construction in civil engineering for dams in North America.

At least one reason for this is anticipated by Justin, Hinds and Creager (p. 782, 1945) who state:

> "Because of the magnitude of some hydraulic fill dams, great publicity has been given to the several slides and construction accidents which have occurred in connection with several of them. As a result, some engineers exhibit a prejudice against that type of construction. As a matter of fact, since 1930 there have been several more or less serious construction accidents in connection with rolled fill earth dams but only one in connection with a

> hydraulic fill dam. There is no sound reason for such prejudice. All that is necessary is for the engineer to appreciate that the hydraulic fill dam is an engineering structure and that it should be given the same competent attention in investigation, design, and construction that one would give any other engineering structure".

Prejudice or not, and notwithstanding the sage comments repeated above, few engineered hydraulic fill structures were built in North America, and western Europe in subsequent years, at least within civil engineering practice. Engineering for hydraulic fill structures was not treated in academic curriculum and the concept developed an aura of lack of safety. This was accentuated in the post 1960's by the increased concern over and research into earthquake induced liquefaction. There was also more competitiveness from rolled fill dams. Following World War II, there was a dramatic increase in the scale and cost-effectiveness of motorized earth-moving equipment which contributed enormously to the feasibility and economy of large earth-moving operations for dams and other engineered fill structures.

While an exhaustive study of the recent literature on engineered fill structures in the Western world has not been undertaken it is our experience that the prejudice identified by Justin et.al. remains intact. The contributions to the 14th Congress on Large Dams in 1982 are characteristic. Question 55 was devoted to materials and construction methods for embankment dams and cofferdams, excluding tailings dams, but otherwise singling out hydraulic fill dams for attention. In a comprehensive General Report on this Question, Penman (1982) is able to devote only a few lines to hydraulic fill in Western practice, noting that it is used to build flood protection dykes in Holland and elsewhere, that it continued as a method of dam construction until the failure of Fort Peck in 1938 and that the technique was used as part of the fill for the High Aswan Dam where the fill as placed had a dry specific weight of 15.2 kN/m^3 which was subsequently increased by vibration to 16.5 kN/m^3. Reference is also made to the more extensive use of hydraulic fill methods in

China (Chang-Tse, 1976) which had obviously evolved under different technological imperatives.

Of the sixty papers published in response to this Question the only paper from a Western source discussing hydraulic fill dams was by d'Angremond et.al. (1982) who advise that hydraulic fill has been used in Holland for dams to a height of about 25 m. Sand that is placed by hydraulic means has a relative density of 50-65% when placed above water and 35-40% when placed under water, which imply 100% and 85-90% of standard compaction respectively. Additional compaction can be undertaken. Construction details are provided.

The paper by Borovoi et.al. (1982), which represents the views of the USSR National Committee on Large Dams, states that the hydraulic fill method is in common use in the USSR and that it is finding increasing applications in hydro engineering. Typical material specification limits and slope inclinations for preliminary design are given. These independent developments will be discussed in more detail below.

As noted by both Justin et.al. (1945) and Penman (1982) the collapse of the Fort Peck Dam constitutes a watershed in the evolution of hydraulic fill methods for dam construction in North America and marks its decline. The failure of Fort Peck Dam was subjected to extensive investigations, most of which are reviewed and referenced by Middlebrooks (1942). The consulting board appointed to investigate the causes of the slide concluded that the slide:

> "was due to the fact that the shearing resistance of the weathered shale and bentonite seams in the foundation was insufficient to withstand the shearing force to which the foundation was subjected. The extent to which the slide progressed upstream may have been due, in some degree, to a partial liquefaction of the material in the slide".

Middlebrooks (1942) emphasizes the following:

> "It can be concluded definitely that the hydraulic fill was not at fault. It was the general opinion of the investigating engineers and other engineers having an intimate knowledge of the project that the fill material performed excellently even under the most difficult circumstances existing during the slide".

While hydraulic fill methods are common in dredge disposal and reclamation, they almost disappeared from the construction of water storage dams in Western practice. It is of value to question whether this has been a matter of prejudice or prudence.

This should not imply that the design and construction of hydraulic fill received no attention elsewhere. As noted earlier, Soviet practice makes much use of the technique with a variety of materials ranging from silts to gravel with a maximum particle size of 100-150 mm. Moreover hydraulic fill applications to engineered structures is finding increasing application in the USSR. In the mining industry, the requirements to dispose of mill tailings in an economic manner resulted in the evolution of the tailings dam which, as mining operations became bigger and bigger, also became very substantial. Today the largest dams in the world in terms of volume are tailings dams and it has become increasingly common for their safety requirements to be the same as for water storage dams.

Another area of activity that has encouraged the development of hydraulic fill technology is the exploration for hydrocarbons in the Beaufort Sea. As explorers for hydrocarbons in the Arctic ventured off-shore in the early 1970's, they encountered ice as a dominant consideration affecting the safety and cost of the exploration system. Ice affects ship navigation and safety, limiting the usefulness year round of drillships. For fixed bottom structures, such as jack-up rigs, ice can exert very substantial horizontal loads that are not tolerated by most conventional rigs. Hydraulic fill islands were perceived as an economic, environmentally sensitive method of providing year round access for off-shore drilling with structures capable of resisting the design ice forces.

As summarized above, the hydraulic fill technology has been abandoned in some areas of civil engineering practice, but has evolved in others. Hence, a re-evaluation of this technology is timely. To facilitate this re-evaluation we summarize in separate sections to follow:

1) the development of hydraulic fill technology in the USSR,
2) the applications of hydraulic fill to tailings dams,
3) the construction of off-shore islands by hydraulic fill methods.

A classification of hydraulic fill structures is presented based on the environment of deposition, and the end product from a geotechnical perspective. Certain specific design and construction issues are central to most hydraulic fill problems. The most important among them are singled out for special discussion in a later section.

2.2 - HYDRAULIC FILL FOR WATER STORAGE STRUCTURES IN THE USSR

Between 1947 and 1973, over 100 hydraulic fill dams with a corresponding total volume of 800 x 10^6 m^3 were built in the USSR. (Hydroprojeckt, 1974). Only a few failures occurred. They happened during construction and were generally not very significant.

More than 10% of the Soviet dams are hydraulic fills. This is a significant number if one considers that the hydraulic fill technique is mainly adopted where alluvial foundations and wide valleys prevail. These conditions are encountered on lowland rivers and such rivers provide only 20% of Soviet hydropower potential (Energomachexport, 1974).

The hydraulic fill technique has been used in the USSR not only for water storage dams, but also for tailings disposal, artificial islands for off-shore oil exploration, reclamation, and fill construction under winter conditions. In addition it has been used as an efficient and economic technique for both diversion of rivers and direct dam construction without diversion. The success of this application is related to the capability of delivering an enormous amount of material in a short period of time which is readily achieved by hydraulic filling. Finally, hydraulic fill is also used for construction on slump-prone foundations, such as collapsible soils, loess, etc. because wetting takes place during the placement of the structure, while the load is increasing. Foundation settlements occur mainly during the sluicing period when the fill material is most deformable (Perevezentseva, 1977).

An evolution of both design and construction methods can be traced in Soviet hydraulic fill technology. Cross-sections adopted in current practice are characterized by only modest permeability contrasts or no contrast at all, no positive cutoffs, and flat slopes. Typical sections are discussed in more detail below.

Techniques to attain high rates of construction are central to Soviet practice. According to Melent'ev (1980), with the appropriate work plan and equipment, hydraulic fill output has reached 300,000 m^3/day on some projects in the USSR. Such high rates of construction require specific equipment such as special dredges and pumps as well as the utilization of free draining granular material. High outputs and high rate of construction result not only in a reduction of cost but also appear to produce denser fill.

Construction of hydraulic fills are also undertaken under severe winter conditions. Development of reliable methods of maintaining holes through the ice for the working reservoir of the dredge, and heating of the faces being excavated, the slurry and the pipelines, provide for stable and economic operations at subfreezing temperatures. Shkundin (1976) points out that the difficulties related to excavation, transport and deposition under winter conditions diminish sensibly with increasing equipment capacity.

Equipment with an output of more than 3000 m^3/h can be operated under winter conditions in Siberia.

2.2.1 - Typical Cross-sections

The zoned or heterogeneous profile contains a low permeability core and granular shoulders achieved by hydraulic segregation, see Figure 2.1(a). The core is not consolidated and, as recognized in early Western designs, exerts high pressure on the shoulders. The thickness of the core is controlled by the dimensions of the pool and the grain size distribution of the borrow material, especially the percentage of fines. Cores are formed when the borrow material has a coefficient of uniformity greater than 3. Zoned dams of this type with impervious cores are not very common in the USSR.

Central zone profiles are more common. This type of profile resembles the zone profile in the sense that it has a core that is less pervious than the shoulders. The difference is that in the central zone profile, the core is not formed by plastic material, the core is not thick and the permeability ratio between the core and the shoulders is only 10 to 100. In the zoned profile this ratio is 100 to 10,000. Central zone profiles are constructed when the borrow material has a coefficient of uniformity between 2 and 3 (Yufin, 1965). Due to restraints of both time and fill volume associated with core consolidation, Soviet hydraulic fill technology moved towards more homogeneous profiles.

The homogeneous profile is characterized by the same grain size distribution throughout the whole section, see Figure 2.1(b). This is attainable when the soil from the borrow area has a coefficient of uniformity less than 2. This type of dam can be built with sand, sandy silt or loess. There is no pool formation during the construction of the homogeneous profile. The slurry is allowed to flow long distances forming flat slopes, say 20-50H:1V. This profile has proved to be effective, especially for dams under 30 m. For higher dams, the volumes required become huge due to the flat slopes that develop.

The mixed profile is composed of part mechanically placed material (dumped or compacted) and part hydraulically filled material (see Figure 2.1c). Usually, the construction starts with mechanical placement of massive shoulders, which provide stability for the dam during construction and operation. Then, the space between the shoulders is filled hydraulically. The mixed profile limits the width of dam and provides extra resistance to failure during earthquakes.

There are many variations of the mixed cross-section. The Playavinyas Dam, for example, has the downstream shoulder placed by trucks while the upstream and central zones were placed hydraulically.

2.2.2 - Borrow Areas

For most, if not all, Soviet hydraulic fill projects, the borrow area is an alluvial deposit in the river channel or a flood-plain deposit which is exploited by dredging, as opposed to abutment borrow areas excavated by hydraulic monitors which was the common procedure in early Western practice.

The factors that determine the adequacy of a potential borrow area are the type of material, the water availability, its position with relation to the dam and the volume of material available. Almost every size of material can be utilized from gravel to clay. However, the utilization of very coarse or very fine materials is more difficult, more time consuming, and more expensive than the use of sandy soils. The difficulty in using coarse materials is related to extraction problems, loss of hydraulic head and high wear of pipelines and equipment in general. According to Melent'ev (1980), the maximum acceptable diameter of particles is about 10 to 15 cm, but such values must be considered in economic terms to justify their utilization. The use of clayey soils lead to the need to invoke additional procedures to allow the material to dry and to adopt low rates of filling, which increase construction time and costs, unless the clay forms balls. Hydraulically deposited clay balls have also been used (Lutovinov et al., 1975). Most hydraulic fill dams in the USSR are built with sandy materials, but there are some dams composed of sandy gravel and clayey soils.

Based on experience, the Soviets have defined limiting grain size curves which are recommended for a preliminary determination of material adequacy at the design stage. The curves are presented by Melent'ev (1980) and Borovoi et.al. (1982) among others. The grain size distribution curves of the materials from North American hydraulic fill dams cover a wider range of values and are less uniform than the limiting grain size curves defined by the Soviet technology.

2.2.3 - Hydraulic Sorting Analysis

Hydraulic sorting refers to the process of deposition of particles of different sizes at different distances from the discharge point. Larger particles tend to deposit soon after being discharged, while small particles can be carried by the flow and deposited further downslope.

The Soviet standard specification SNiP-11-53-73 recommends consideration of sorting if the borrow material has $D_{60}/D_{10} > 2.5$ and/or $D_{90}/D_{10} > 5$; although, actually, hydraulic sorting starts to be observed for $D_{60}/D_{10} = 1.3$ to 1.6 if the beach is long enough.

The hydraulic sorting is usually forecast by Melent'ev's method. This method (Melent'ev et. al., 1973) is a semi-empirical method based on a large set of experimental data, hydraulic flow laws and some statistical analysis which determines the average grain size distribution at any distance from the discharge point.

2.2.4 - Construction Technique

The Soviets have several different techniques of hydraulic fill placement. However, since the 1950's they have been using almost exclusively trestleless pipelines with extendible single point discharge, see Figure. 2.2. In this case the slurry is discharged through the end point of a pipeline that lies directly on the downstream slope surface. The slurry flows in the upstream direction forming a very flat wave resistant slope. The position of the end of the distributing pipeline has to be changed from time to time to form an even slope. To achieve this, the pipeline is set up with a quick joint system so a crane can install and remove segments of pipe without stopping the slurry discharge. Little foundation preparation, except for removal of top soil, is involved. In addition, clean layers of gravel are treated or removed.

The rate of filling is kept within allowable values in order to assure stability of the structure. Excessively high rates of filling can cause instability of the structure due to a high phreatic level in the shoulders and/or piping at the point where the phreatic surface merges on the external slope. This is especially critical when the foundation has a low permeability. An example of such instability occurred during the construction of the Bratsk dam (Tarasenko, 1970).

In most Soviet dams within the river channel, a toe drain is built on the downstream side. For dams on flood-plains, the most common drainage system is a pipe drain. In some cases, inverted filters are installed in the lower part of the downstream slope. The drainage system also usually includes downstream relief wells.

With this technology, the Soviets have been successfully applying hydraulic fill techniques to a variety of foundation conditions, site characteristics, using a number of different materials.

Excellent performance of hydraulic fills has been reported even under relatively severe seismic conditions (Volnin and Ivanovskaya, 1977).

2.3 - HYDRAULIC FILL AND TAILINGS DAMS

While there are invariably many options other than hydraulic fill for the construction of a water storage structure, in the case of the disposal of mine and mill waste there are few. Many mine and mill operations produce a wet waste stream and for all but the shortest distances, it is usually substantially more economical to transport solids hydraulically than by truck, conveyor, or other dry handling methods. Therefore hydraulic fill methods are naturally attractive to build retention structures of and for tailings.

Some of the largest earth structures in the world are tailings dams. For example, the tailings dam at Phalaborwa in South Africa is expected to have a volume of 162 x 10^6 m^3 at completion which is to be compared with the Tarbela Dam, the largest (volume) earthfill in the world, having a volume of 142 x 10^6 m^3. At the Syncrude oil sands project in Canada, tailings are stored in an out-of-pit tailings pond until a future date when they will be introduced into the mined out pit. Approximately 267 x 10^6 m^3 of sand and 318 x 10^6 m^3 of thick sludge will require storage in the Syncrude tailings pond. To accommodate the almost 600 x 10^6 m^3 of solid waste, approximately 18.5 km of hydraulic fill dykes, ranging from 27 to 82 m in height, will have to be constructed. At completion, the tailings pond will have a surface area of 17 square kilometers. These and other mining projects that utilize hydraulic fill, involve materials handling on a grand scale in both an economic and industrial environment that differs from that of water storage dam construction.

Tailings are of no direct economic value and therefore there is a pervasive pressure to find the most economical, acceptable way to dispose of the tailings. The construction of a tailings dam is usually carried out by the mine operators with the height of the dam being increased as required to provide the needed waste storage. This has advantages when compared with water storage dams which are usually completed in a short period before filling and which are usually constructed within a less flexible contractual arrangement than tailings dams. However, construction control is usually poorer and failures arise due to misapplication of sound construction practice. Economic considerations lead to the use in construction in most tailings dams of the coarser fraction of the tailings instead of importing more costly borrow material. Another important distinction is that the bulk of the material stored is not water but loose, sometimes impervious, and potentially liquefiable, tailings.

The upstream method is the oldest, simplest and most economical method of tailings dam construction, see Figure 2.3. It appears to have evolved empirically over many decades without much benefit of geotechnical input. The downstream shell is raised in increments as needed. In North America, for many years tailings were most commonly discharged by spigotting but single point discharge and cycloning were not unknown. A beach was formed and good construction practice minimized the pond level and kept it well off the beach. The downstream face of the dam was often at or near the angle of repose. On reflection, it appears that the structural element of the dam was really the beach which, because of its layered structure and connection to the clean sand shell, could be assured to be fully drained. Provided no water entered into the sand shell, and the starter dam did not block drainage, the low factor of safety of the face was not too critical. Deeper instability through the beach would be critical.

While many upstream tailings dams were constructed in a satisfactory manner, failures were also common due to a variety of geotechnical and construction reasons Following the catastrophic failures at Aberfan, El Cobre, and Buffalo Creek, it became clear that geotechnical engineering had much to contribute to ensure the safety of mine waste disposal systems. It was increasingly recognized that the original upstream method did not result in structures of adequate safety and departures from this traditional structure were advocated. This resulted in increased use of centreline or downstream construction procedures on the one hand, see Figure 2.3, and major modifications in upstream construction on the other.

Downstream methods have the advantage that as the dam is raised, it is not underlain by previously deposited tailings. With the provision of adequate compaction and seepage control, safe dams to substantial heights can be achieved. However, the volume of coarse tailings required for embankment construction is about twice that required in the centreline method and many tailings streams are inadequate in this regard. The configuration of many valleys or other storage areas may also be unsuited to provide abutment ties for downstream construction.

The centreline method is a variant of the downstream method, but instead of the crest moving downstream as the dam is built, it rises vertically. Placement and compaction control can be exercised as required. Provided that the tailings stream is suitable, construction can proceed quickly using coarse-grained cyclone underflow. Underdrainage systems can be installed as the dam is built, thereby controlling the line of saturation. One disadvantage is the large volume of sand needed for construction which is often a problem in the early stages of mine production. Therefore a higher starter dam of imported fill is needed to initiate centreline construction.

From the 1970's on, the design and construction of tailings dams has attracted a substantial literature. Some major references are Blight in (Morgenstern, et al, 1977), Klohn (1981), and Vick (1983). Others are noted by Morgenstern (1985). It is not intended to address details of tailings dam design and construction here.

Notwithstanding the contributions of geotechnical engineering to the rational design of tailings dams, failures still occur. The better known examples are the Mochkoshi Dams in Japan (1973), the Tyrone Dam in the USA (1980), two tailings dams in Chile (1985) and the Stava tailings dams in Italy (1985). It is of interest to note that all of these structures were constructed by the upstream method. The authors are not aware of any collapse of tailings dams constructed by the centreline or downstream method.

This should not be construed as a blanket condemnation of upstream construction. The number of tailings dams constructed by the upstream method is much greater than by the downstream and centreline methods combined and it is therefore reasonable that the statistics of failure reflect this. Safe structures can be built by the upstream method provided that their design and construction respect geotechnical principles. Modern upstream construction employs flatter downstream slopes than their traditional predecessors, beaches are made wider and compaction is sometimes employed.

Experience with the use of hydraulic fill in the construction of tailings dams has revealed certain characteristics that bear on both design and performance and that are of interest for any application of hydraulic fill techniques. The most significant are:

1) Particle size separation
2) Drainage measures
3) Compaction
4) Earthquake resistance

2.3.1 - Particle Size Separation

The common methods of tailings disposal and dyke construction include beach deposition by either single-point discharge or multi-point spigotting, on-dam cycloning and stationary cycloning with subsequent re-handling. The grain size distribution as deposited depends on the method of placement and therefore this aspect of the composition of the resulting fill is, to some degree, controllable.

Abadjiev (1985a) summarizes methods for determining the grain size separation along beaches composed of spigotted materials. Blight and Bentel (1983) have also developed a theoretical relation for the particle size sorting that occurs on a hydraulic beach and demonstrated at least a consistent fit with observations of beaches in gold and platinum tailings. With regard to cycloning, multiple pass cycloning can be used to control the fines content of the underflow used for fill construction. This has a significant bearing on both permeability and earthquake resistant properties and, as noted by Troncoso and

Verdugo (1985), it may be worthwhile to invest in the necessary cycloning efforts required to lower the fines content of a fill material.

It will be stressed in a subsequent section that our ability to forecast and control grain size separation is still limited and a greater understanding of this process is needed. Nevertheless, it is possible to increasingly regard hydraulic fill in some applications as an engineered fill with controllable properties.

2.3.2 - Drainage Measures

Modern tailings dam design and construction relies on internal drainage measures to assure integrity of the structure. Drains are needed to control both construction water and long-term seepage. The significant investment associated with various drainage measures was noted by Morgenstern (1985).

Drains are readily incorporated into downstream construction. The Perez Caldera No. 2 dam in Chile (Griffin et al, 1983) is an example of a comprehensive internal drainage system in a dam that has been designed with special attention to seismic stability.

Internal drains are equally desirable, if not more so, in upstream construction. Examples of internal drains used in the large tailings structures associated with the oil-sand industry in Canada are summarized by Morgenstern (1985). Abadjiev (1985b) shows that the provision of an extensive drainage tongue upstream of the starter dam can improve the stability of upstream construction. The tongue projection, shown in Figure 2.4, is built of free draining material like the starter dam. Its length is equal to the distance from the starter dam to the decant pond at the initial stage and it has an inclination to the pond no smaller than that of the beach, thus ensuring that the coarser fractions are deposited on the tongue. This enhances both seepage control and stability in an economical manner.

Inventive use of drainage measures will extend the applications and economy of hydraulic fill construction.

2.3.3 - Compaction

The designer of a hydraulic fill structure, at least above water, is not restricted to accept the density of the fill as deposited. Clearly both downstream and centreline construction lend themselves to compaction. It is not so well known that upstream construction is also readily compacted.

To achieve compaction, the oil sands industry had adopted the hydraulic cell method of deposition, which after compaction, achieves a relative density of about 75% quite economically (Mittal and Hardy, 1977). Additional details on the hydraulic cell method and subsequent compaction are given by Handford et al. (1982).

2.3.4 - Earthquake Resistance

The catastrophic failures of several upstream-constructed tailings embankments in Chile in 1965 highlighted the limited earthquake resistance of tailings dams constructed by upstream methods (Dobry and Alvarez, 1967). This is further emphasized by liquefaction of the Machikoshi tailings dam (Ishihara, 1984) and failures again in Chile following the March, 1985 earthquake (Troncoso, 1988).

That some of these structures were intrinsically unsafe is clear. Nevertheless many upstream hydraulic fill dams have successfully survived major earthquakes. A particularly interesting example is the Dashihe tailings dam which was subjected to the Tangshang earthquake in 1976.

A cross-section of the Dashihe dam is shown in Figure 2.5 (Central Research Institute of Building and Construction, 1987). The area in which the tailings dam was located experienced a magnitude 7.8 shock on July 28, 1975 and another of magnitude 7.1 some fifteen hours later. The dam was located 40 km and 15 km from the epicenter of these two shocks respectively. In addition there were numerous aftershocks with magnitude greater than 5.

As a result of the earthquake, some cracks developed in the downstream face and on the back, near the pond, there were sand boils, waterspouts and a fissure zone. Nevertheless, the dam served its intended function and continues to do so today.

The circumstances controlling acceptable behaviour of an upstream tailings dam during an earthquake remain enigmatic. Experience indicates that the construction method is not intrinsically unsafe but further studies are needed to isolate those details of construction procedure, fill material, and drainage control that result in safe structures.

2.4 - HYDRAULIC FILL AND ARTIFICIAL ISLANDS

Hydraulic fill placed under water behaves differently than fill placed above water. It tends to assume flatter slopes, be more sensitive to the presence of fines and end up in a looser state on average than hydraulic fill placed sub-aerially. Equilibrium slopes and density achieved in-situ by placement alone are very dependent on underwater fill placement methods.

Hydraulic fill underwater has been used for number of applications such as earth dam and embankment construction, expansion of industrial and harbour facilities and the development of man-made islands. For example, underwater fill was placed and compacted as part of the High Aswan Dam and underwater fills were constructed as part of the Yonkers, N.Y., sewage treatment plant. Johnson et al. (1972) remain a valuable reference in this regard for experience through to the early 1970's and Whitman (1970) provides a comprehensive review of experience with hydraulic fills used to support structural loads. Not all of these latter cases deal with underwater placement. Ishihara et al. (1981) provide information on the characteristics of the fill used in the construction of Owi Island No. 1 in Tokyo Bay.

The greatest expansion in recent times in the underwater placement of hydraulic fill for engineered structures arose as a result of accelerated hydrocarbon exploration efforts in the Canadian Beaufort Sea. Offshore exploratory drilling in the Beaufort Sea began in the early 1970's with an extension of onshore techniques into very shallow water and the use of drillships in deeper water. Due to severe ice conditions, drillships were limited to a restricted drilling season and there were incentives to maintain drilling year round. The initial artificial island structures pioneered by ESSO Resources Canada Ltd. were extended to deeper and deeper water. However, these structures were waterline penetrating sacrificial beach islands with gentle side slopes of 10:1 to 15:1. This concept was impractical in water depths greater than about 20 m due to both economic and construction schedule considerations.

Dome Petroleum Ltd., who had leases in deeper water, took the lead in developing steep-sided submarine berms that would support a vertical sided, waterline penetrating caisson designed to resist the environmental forces and still provide workspace for the exploration drilling. While originally conceived as a temporary structure, this concept is adaptable to a permanent structure and is under consideration for use when hydrocarbon production starts.

The first topside unit comprised a multiple concrete caisson system which was subsequently replaced on later islands by various forms of steel drilling caissons that were more readily deployable. This constituted a major advance over sacrificial beach islands in terms of reduced fill volumes, minimizing the effects of wave erosion, enhanced mobility and pre-assembly of at least parts of the drilling system requirements. Sacrificial beach and waterline penetrating caisson systems are compared in Figure 2.6.

Artificial island construction began in 1972 and 1973 in only a few meters of water. Fill quantities were small, approximately 50,000 to 200,000 m^3 per island, and construction methods varied from trucks hauling gravel from onshore onto the winter ice to a clamshell barge placing local material or placement with a cutter suction dredge. Gravel is in short supply in this region and its availability for larger structures would ultimately be proscribed.

From these beginnings, the sacrificial beach island concept evolved, which reached its climax in 1978-79 with the construction of the Issungnak island in 19 m of water. This island was 100 m in diameter, and required 4.1 x 10^6 m^3 of fill which was obtained mainly by using stationary suction dredges in an adjacent sea bottom borrow pit. The fill exhibited an average D_{50} of 200 microns and a fines content of 15%. All fill material was placed at the site by pumping the sand slurry from the dredges to discharge barges at the island through floating pipelines (Boone, 1980). Final construction slopes stabilized at 10:1 but they may have been re-worked later as a result of storm erosion.

Mass movement from sacrificial beach islands are primarily due to storm-induced erosion. However, it has been suggested by Crooks et al. (1985) that there was also storm-induced liquefaction at another island (Alerk) which was also constructed of hydraulic fill containing a substantial proportion of fines.

The Tarsiut N-44 island constructed in 1980-81 marked a major departure from the sacrificial beach island concept. It was the first island to be built outside of the landfast ice zone and hence large horizontal ice forces had to be contemplated in design and it was the first caisson retained exploration island constructed in the Canadian Beaufort Sea. The island was in 22 m of water and had a soft clay foundation which was sub-cut to a depth of about 2.5 m - 3 m and backfilled with sand.

Sand berm construction which was to a height of 17.5 m above the sea floor was achieved in about two months. Much of the material was dredged from a distant borrow and transported to the island in split-hull dump barges with individual capacities of 1500 m^3. This material, when dumped, was controlled by concentric bunds with sideslopes of between 5:1 and 6.5:1 which were constructed by pumping hydraulic fill through a discharge pipe mounted on an anchored barge filled with borrow (Fitzpatrick and Stenning, 1983). The specifications for the fill were D_{50} greater than 250 microns and less than 10% fines. Except for construction error when some clay balls were introduced into the fill for the caissons, the borrow readily met these specifications. About 1.8 x 10^6 m^3 of hydraulic fill was used at Tarsiut, including fill for the caisson. Cone testing indicated relative densities of about 60% which was in excess of design requirements. While there were some construction problems, Tarsiut performed as intended. It demonstrated that steep containment slopes could be constructed and that medium dense fills were attainable by bottom dump placement methods. The construction technique is illustrated in Figure 2.7 and the completed berm with the multiple concrete caisson placed on top is shown in Figure 2.8(a).

The Uviluk site constitutes the next major advance in artificial island experience. The site was characterized by a 31 m water depth and a competent seabed foundation. At Uviluk, the multiple concrete caisson system adopted at Tarsiut was abandoned in favour of a single steel drilling caisson system (SSDC) which had many advantages in terms of deployment and overall economics. A comparison of the two systems is shown in Figure 2.8 (Mitchell, 1984).

The submarine berm at Uviluk, comprised 2.1 x 10^6 m^3 and the sand had a D_{50} of 320 microns and less than 5% fines. Fill placement was similar to Tarsiut but with improved quality control. The finished berm had nominal side slopes of 7:1. Details of caisson set-down and subsequent deformation of the hydraulic fill are beyond the scope of this presentation. Suffice it to say that the settlement of the hydraulic fill and the lateral deformations under load were generally as forecast.

Cone penetration testing of the berm revealed a substantial sensitivity to what might be regarded as relatively minor variations in material type. This was subsequently studied in more detail by (Berzins and Hewitt, 1984) who discovered a very substantial variation of cone resistance at a given depth with relatively modest variations in the fines content in otherwise comparable fills. Their correlation is shown in Figure 2.9.

Following the success of Uviluk, and the increased understanding of the factors controlling placement density, as well as improved quality control procedures, the construction of substantial submarine berms appeared to be coming routine. This proved not to be the case.

The Nerlerk sand berm was initiated in 1982 in 45 m of water. From the point of view of hydraulic fill operation, the construction at Nerlerk was made more difficult by:

1) the increased water depth and hence the added pressure to build slopes steeper and faster than before to meet the set-down schedule,
2) the need for the almost exclusive use of a deep suction exploration and pipeline placement system to meet the schedule,
3) the need to exploit a local marginal borrow source to support the above construction operation, and,
4) the uncertainty as to the in-situ densities which one could expect using these materials and placement techniques.

Approximately 4 x 10^6 m^3 of fill were placed, almost completing the berm, when five major slope failures occurred. The site was ultimately abandoned. Details are given by (Mitchell, 1984).

Most of the fill at Nerlerk was deposited by a nozzle from a relatively stationary placement barge which was connected by floating pipeline to a deep suction dredge. At the time of construction it was not recognized that construction methods could have such an overwhelming influence on depositional density. Figure 2.10 from (Mitchell, 1984) compares nozzle deposited with hopper dredge deposited material of comparable composition to illustrate this point. A hopper dredge utilizes a bottom valve release system.

The Nerlerk failures emphasize our limited understanding of the interaction between in-situ densities and placement technique.

Following Nerlerk additional islands were built and operated with only minor problems, but never in water depths exceeding about 30 m. One sand-filled caisson system was subjected to a sustained high ice loading in 1986. This ice loading is of a cyclic nature and pore pressures and movements in the core fill continued to increase until the ice movement ceased. The behaviour approached the limits of serviceability and underlines our limited ability to calculate deformation of saturated sand fill under horizontal cyclic loading. Details of subsequent analyses of this event have not yet been published.

Approximately 40 x 10^6 m^3 of hydraulic fill have been utilized in artificial island construction in the Canadian Beaufort Sea. The program has not been without mishap but the geotechnical achievements have been substantial. However, there is much more to be understood before the design and construction of large sub-aqueous hydraulic fill become routine.

2.5 - CLASSIFICATION OF HYDRAULIC FILL STRUCTURES

Hydraulic fill has several applications in different areas that involve professionals with diverse backgrounds. To facilitate discussion and to emphasize some of the factors controlling the behaviour of hydraulic fill structures a classification of such structures based on the physical phenomena involved rather than the type of utilization is proposed. This classification is shown in Figure 2.11.

The geotechnical behaviour of the fill is much affected by the environment of deposition, the mechanics of deposition and the processing of the fill. Different end products result, not only in terms of geotechnical material but also in terms of geometry.

considering these factors, the proposed classification is based on structures with similar geotechnical considerations.

The first distinction in the classification is made between sub-aerial (above water) and sub-aqueous (under water) deposition. As noted previously, sub-aqueous hydraulic fills, prior to any compaction, tend to be looser and more variable than comparable fills deposited sub-aerially.

The next distinction that is made is whether or not a beach is created. Beach deposition involves a regular segregation pattern with the coarser fraction being deposited adjacent to the discharge point and the finer fraction being deposited farther away.

For sub-aerial non-beach deposition, the final component of the classification reflects the end product in terms of degree of homogeneity. Some examples are given, but this is not intended to be an all-inclusive list.

Rock-infilling refers to the sluicing of sand into the voids of rockfill as was done at the High Aswan Dam. Patterned refers to the filling of cells to construct a structure. This method has been adopted in the USSR to build homogeneous dams using heterogeneous borrow. Random refers to the type of filling commonly used in land reclamation and the hydraulic backfilling sometimes adopted underground in mines (Mitchell et.al., 1975). Thickened discharge refers to the procedures developed by Robinsky (1979) to inhibit segregation by adjusting the placement water content so that a stable pile can be achieved, at least for purposes of waste disposal.

With regard to sub-aerial beach deposition, the distinction is made between whether the resulting structure is composed of homogeneous fill or not, as indicated by a coefficient of uniformity, say, less than 3 for homogeneous fill. The final component of this portion of the classification refers to the geometry of the end product and design and construction aspects such as the need to ensure separation of fines, pool design and type of drainage system required.

A contained heterogeneous fill refers simply to the beaching of the hydraulic fill between containment dykes that might be of rockfill. The classical sections of tailings dams, shown in Figure 2.3, are examples of zoned structures.

Homogeneous fill structures arise either from the deposition initially of fill material without fines or heterogeneous material with fines being removed during construction. An example of the latter case is current USSR practice in construction homogeneous hydraulic fills.

With regard to sub-aqueous deposition, the distinction is also made whether a beach is created or not. With regard to non-beach deposition the free drop of the slurry (H) exercises an important influence on the end product and a further distinction is made whether the hydraulic fill is placed gradually or dumped. Some zonation is possible by mixed construction modes.

2.6 - ISSUES IN DESIGN AND CONSTRUCTION OF HYDRAULIC FILLS

Based on the perspective presented in the previous pages, several issues that are central to improved design and construction of hydraulic fill are highlighted in the following. The issues focus on concerns that affect the reliability of utilizing hydraulic fill for an engineered structure. Other important subjects such as construction methods, consolidation of hydraulically deposited fines, and the use of hydraulic backfill in underground mining are not considered here.

2.6.1 - Role of Laboratory and Field Experiments

It is of interest to review the elements of current practices when dealing with rolled fills. After the potential borrow area is delineated by site investigation, disturbed samples are taken and compacted in the laboratory. The compaction energy used is approximately compatible with the energy to be used in the field to produce samples with similar fabric. Laboratory tests with these samples are used as a basis for material characterization and the establishment of construction specifications. These parameters enter directly into embankment design and constitute a basis for field quality control. This is a straight forward procedure well supported by years of experience.

Hydraulic fill design poses more difficult questions right at the outset. Even knowing the borrow material, segregation and loss of fines make the fill material different than the borrow. Furthermore, hydraulic deposition creates characteristic fabric, micro and macro layering, which cannot be reproduced in a conventional soil mechanics laboratory. Flume tests probably can create similar depositional conditions, but it is still not very clear how the fill density can be evaluated from these tests.

While the potential contributions of laboratory and field experiments are yet to be worked out, any generalizations will ultimately be limited by scale effects. The observations of Valore and Giglio (1985) are useful in this regard. They note that the hydraulic deposition of dredged materials within containment basins give rise to marked spatial variations in the physical and mechanical properties of the resulting soils. These heterogeneities, in this case, govern the progress of consolidation of the hydraulic fill.

The design of hydraulic fills usually focus on geometry, lay-out and specifications for construction control based on experience. Stability and seepage analysis are done using assumed values that may depart significantly from the actual values that will exist within the fill. However, as the Nerlerk failures indicate, this can be a hazardous process. The resolution of these problems resides at the interface between sedimentary mechanics and soil mechanics and some combination of theory, laboratory experiments, and field scale studies should prove fruitful.

The most ambitious developments in this regard are the relations developed from experience in the USSR with the construction of hydraulic fill dams and which can be applied to tailings dam design. This work is not readily accessed in the Western literature but Abadjiev (1985a) provides a convenient starting point. Empirical correlations describing particle sorting in a beaching operation are described. Correlations also exist between grain size distributions, subsequent compressibility and shear strength. It is clearly easier to make observations and confirm relations for subaerial deposition but, as Ogawa (1969) has shown, even the problem of dispersion of dumped sand from a hopper-barge is amenable to analysis.

So far, laboratory experiments appear to have made only limited contributions to hydraulic fill studies. Blight et al. (1985), Blight (1987), Smith et al. (1986) and Fan and Masliyah (1990) have confirmed by both laboratory flume studies and field investigations the conclusions of Melent'ev et al. (1973) that the profile of a dam beach can be represented by a single dimensionless master profile. As a result, the profile and composition of a beach could be forecast. However, the study by Wates et al. (1987) did not support the concept of master profile.

We have reviewed a number of flume based experiments, mainly unpublished, whose objectives were to forecast beach angles and fill properties as a function of certain depositional parameter such as composition of fill and rate of deposition. The results from various studies have generally been disappointing in that few general trends consistent with field behaviour have been observed. As a result such experimental techniques are not used much. However we are of the view that better-controlled experiments are needed to evaluate the potential of flume studies in hydraulic fill.

A flume facility has been constructed at the University of Alberta. The intention is to study the effect of deposition on the fill characteristics such as slope, grain size distribution, density and shear strength. The tests are carried out under uniform one-dimensional deposition at the discharge point. In order to achieve these conditions sand is fed at controlled flow rates to a water stream. The slurry goes through a vertical flexible pipe that feeds a specially designed spreader. Initial results are just being processed. These flume tests are described in detail and the results are presented in the next Chapter.

2.6.2 - Characterization of Hydraulic Fills

Some hydraulic fills stored as mineral waste or used in reclamation are fine-grained. However, most materials used in engineered fill structures are cohesionless soils and are much simpler to characterize than clays. But cohesionless soils traditionally are regarded as hard to sample and inspect. As a result the characterization of cohesionless soils in routine practice is actually more primitive than that of cohesive soils.

The geotechnical behaviour of a cohesionless soil is dominated by its composition, its in-situ density, its structure and the stresses acting on it. These are listed in more or less generally increasing difficulty of accessibility.

Drawing on the tradition of routine foundation engineering, the difficulties associated with realistic geotechnical characterization of cohesionless soils are circumvented by reliance on various in-situ tests, particularly the Standard Penetration Test (SPT) and the Cone Penetration Test (CPT). Interpretations of soil behaviour range from purely empirical such as for the SPT to interpretation supported by a theoretical base (e.g., Been et al., 1986). It should be widely understood that no such test is perfect and that they are all influenced to various degrees by progressive failure, rate effects, lateral stresses and operational details. This is not the place to enter into a discussion of the arcane world of penetration testing, its corrections and interpretations except to comment that improved engineering of hydraulic fill will require better undisturbed sampling and direct measurement of the in-situ density.

Ishihara (1985) has outlined recent developments in undisturbed soil sampling and it is to be hoped that these techniques will be adopted more extensively, at least on major projects. There have also been major developments in the use of down hole nuclear techniques for the direct measurement of density. The paper by Plewes et al. (1988) is particularly noteworthy in this regard. Continuous downhole density logging was undertaken with a petroleum industry gamma ray density tool to evaluate the in-situ density of a 90 m high tailings dyke. Measurements were possible beyond the depth where SPT data loses meaning. Careful comparisons were made with undisturbed samples and the accuracy reported is impressive. The capacity for continuous direct density logging is a significant contribution to the evaluation of liquefaction potential.

2.6.3 - Stability and Deformation of Hydraulic Fill

Where hydraulic fills are deformed under fully drained conditions their strength and deformation properties are usually unexceptional and amenable to determination by current laboratory testing techniques. Experience indicates that drained stability can be evaluated in a straight forward manner and that both vertical and lateral deformations can be assessed reasonably using methods of non-linear analysis that are common in practice today. The behaviour of hydraulic fill during undrained loading is more problematical.

Undrained loading can arise due to rapid construction, foundation deformation, earthquake and wave effects. The on-going intense studies into liquefaction and cyclic mobility are central to the evaluation of undrained stability and deformation of hydraulic fill.

The articulation of Steady State Line concepts (e.g., Poulos et al., 1985a) provides a fundamental starting point. Only specimens representative of composition are necessary to define the steady state line. Provided the in-situ void ratio is known, the undrained shear strength of the soil in the steady state is known and the stability at the steady state can be evaluated with ease. As a design concept, this takes the view that if the soil can liquefy it will, and the results constitute a bound on the stability assessment. Poulos et al. (1985a) advocate piston sampling for the in-situ void ratio determination but, as noted previously, nuclear methods also have considerable promise. The role of initial fabric on affecting whether stress paths to steady state are much influenced deserves attention.

Design based on the steady state strength of a sand is like designing with the residual strength of a clay; there are circumstances where it is the appropriate strength, but if a higher peak resistance appears available, one is obliged to inquire whether the loading mechanism will exceed the peak strength or not. Given the complexity of the loading mechanisms under consideration and the strain-weakening characteristics of some hydraulic fills, this is a difficult question to answer and is the subject of much active research.

For monotonic undrained failure Sladen et al. (1985a) have proposed the existence of a collapse surface defined by the peak undrained shear strength of a sand at the same void ratio but at different initial confining stresses; see Figure 2.12. This collapse surface is essentially independent of stress path, at least for the limited data available at the time of publication. For liquefaction to occur the soil state has to reach the collapse surface and the shear stress must exceed the steady state shear strength. (Sladen et al., 1985b) applied the collapse surface concept to the analysis of the Nerlerk berm slides with encouraging results. Additional experience is needed with this fertile concept.

For the evaluation of stability under cyclic loading, Poulos et al. (1985b) proposed a methodology that invokes an evaluation of the strain associated with the cyclic loading of the design earthquake and assessment of the strain required to trigger liquefaction and thereby assess whether peak strength is exceeded. The logic of this analysis has wide application to all types of hydraulic fill structures and it merits detailed study.

Sladen et al. (1985a) speculated that the collapse surface also constitutes a bound to the influence of cyclic loading and that liquefaction could occur if cyclic loading took the soil state to the collapse surface. It is intriguing to speculate whether the collapse surface is the same as the "line of phase transformation" proposed by Ishihara et al. (1975) to explain characteristics of the undrained cyclic loading of sands.

Even if collapse is avoided, large deformations may develop during undrained loading that threaten the serviceability of the earth fill structure. Seed (1987) has cautioned that large deformation may precede the mobilization of steady state strength and could control the design. The previously mentioned response of an offshore structure to cyclic ice loads is an apt example. The ability to forecast large undrained deformation of hydraulic fill is still limited but fortunately the subject has become the focus of much research.

2.6.4 - Improvement of Hydraulic Fills

Although the collapse of the Ft. Peck Dam contributed to the decline in usage of hydraulic fill in North America, it should be noted that hydraulic fill was ultimately used to complete the dam. However, this fill was compacted by crawler tractor with relative densities increasing from about 60% to 80-90%. This and other examples of hydraulic fill compaction by machine are summarized in Johnson et al. (1972). Mittal and Hardy (1977) provide a comparable example of machine compaction used in tailings dam construction.

Densification can also be achieved by blasting and pneumatic methods. Compaction is not limited to sub-aerial access but can also be undertaken subaqueously as well. The sea-bed compaction associated with the Oosterschelde project is noteworthy in this regard (Pladet, 1978).

The improvement of hydraulic fills is not limited to densification alone. The incorporation of geosynthetics for purposes of drainage, material separation, or reinforcement provide options that are a challenge to the imagination of designers and constructors alike.

2.7 - CONCLUSIONS

Although hydraulic fill declined in use for water retaining structures in North America about 40-50 years ago, it continued to evolve as a technique for engineered structures elsewhere and in other fields of application. Extensive use has been made of hydraulic fill for dams in the USSR, for tailings dams in many countries, for sea dyking in The Netherlands and for exploration structures in the Canadian Beaufort Sea, to single out some of the more substantial areas of application. The evolution of hydraulic fill technology has been fragmentary and it is timely to initiate a new synthesis of both theory and practice in order to evaluate afresh the potential for engineered hydraulic fill structures.

The role of experiment into the depositional processes as they affect geotechnical behaviour is not yet clear. Some of the more promising findings have been identified. Other key issues addressed that bear on design and construction of hydraulic fill are in-situ geotechnical characterization, the analysis of stability and deformation of hydraulic fills, and the improvement of hydraulic fill properties. Enhanced understanding and experience in each of these areas will lead to more reliable hydraulic fill design and construction.

However, as previous examples indicate, the behaviour of hydraulic fill is much influenced by the placement process. Hydraulic fill technology is an inter-disciplinary activity. The geotechnical engineer will have to collaborate with the hydraulic engineer and the marine construction industry in order to take full advantage of the economies offered by hydraulic fill structures.

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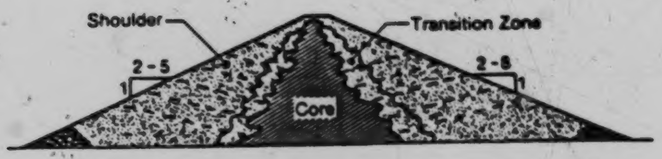
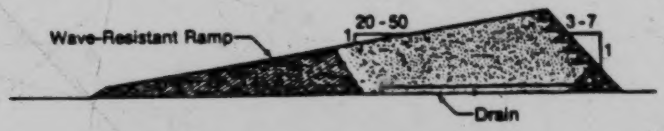
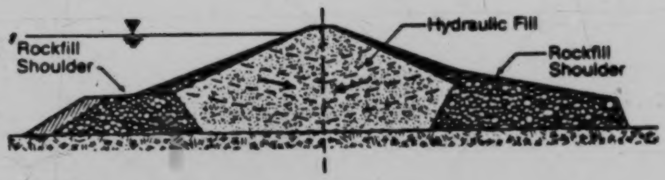

Shoulder
Transition Zone
2 - 5
1
2 - 8
1
Core

(a) Zoned Profile


Wave-Resistant Ramp
20 - 50
1
3 - 7
1
Drain

(b) Homogeneous Flattened Profile


Rockfill Shoulder
Hydraulic Fill
Rockfill Shoulder

(c) Mixed Profile

FIGURE. 2.1 - Typical Soviet hydraulic fill cross sections

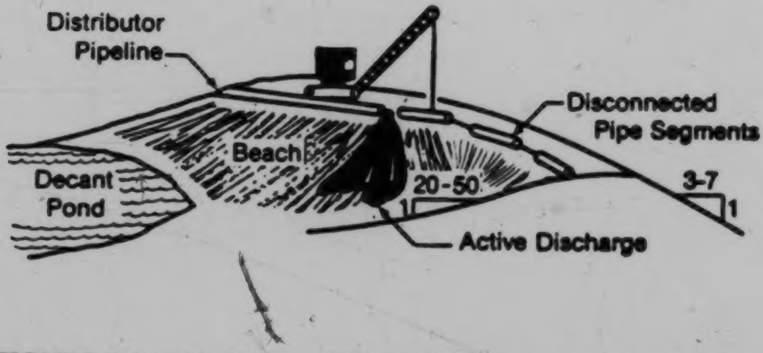
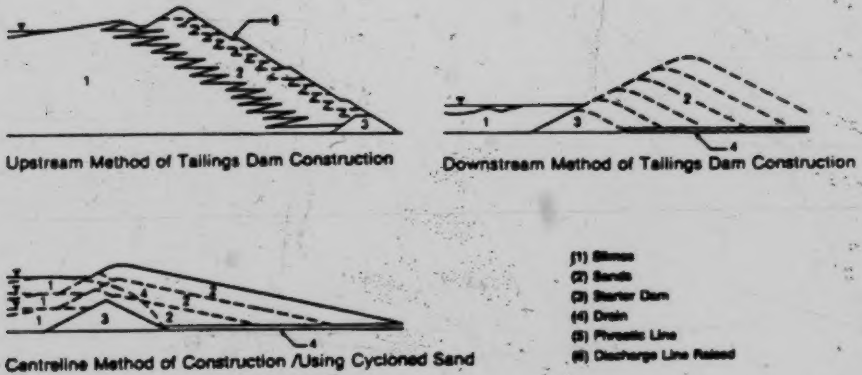

Distributor Pipeline
Disconnected Pipe Segments
Beach
Decant Pond
20-50
1
3-7
1
Active Discharge

FIGURE 2.2 - Fill placement by pipeline and single point discharge



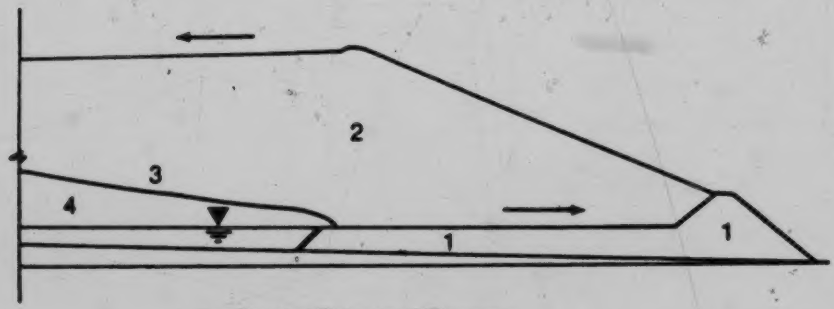
Upstream Method of Tailings Dam Construction

Downstream Method of Tailings Dam Construction

Centreline Method of Construction /Using Cycloned Sand

(1) Slimes
(2) Sands
(3) Starter Dam
(4) Drain
(5) Phreatic Line
(6) Discharge Line Raised

FIGURE 2.3 - Tailings dams construction methods



1 - Starter Dam and Tongue
2 - Beach Deposited Material
3 - Phreatic Line
4 - Decant Pond at the Initial Stage

FIGURE 2.4 - Drainage for upstream construction (modified after Abadjev, 1985b)

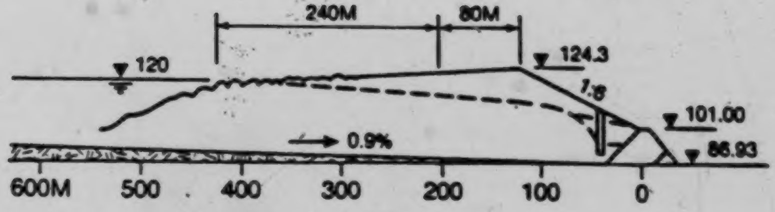
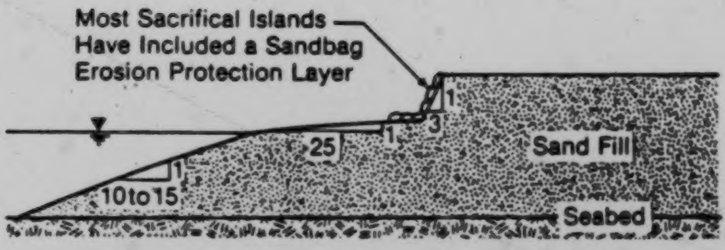
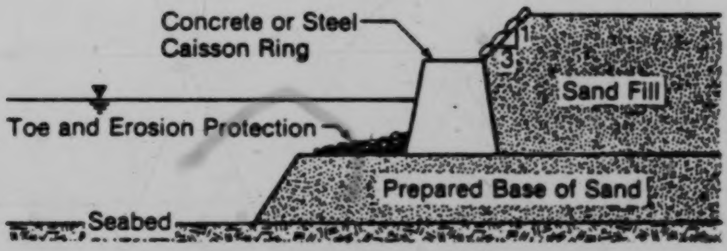

240M
80M
120
124.3
1:8
101.00
86.93
0.9%
600M
500
400
300
200
100
0

FIGURE 2.5 - Dashihe Dam


Most Sacrifical Islands Have Included a Sandbag Erosion Protection Layer
1
3
1
25
1
10to15
Sand Fill
Seabed

Sacrificial Beach — Surface Piercing Island


Concrete or Steel Caisson Ring
1
3
Sand Fill
Toe and Erosion Protection
Prepared Base of Sand
Seabed

Caisson Retained Island

FIGURE 2.6 - Concepts for large artificial islands

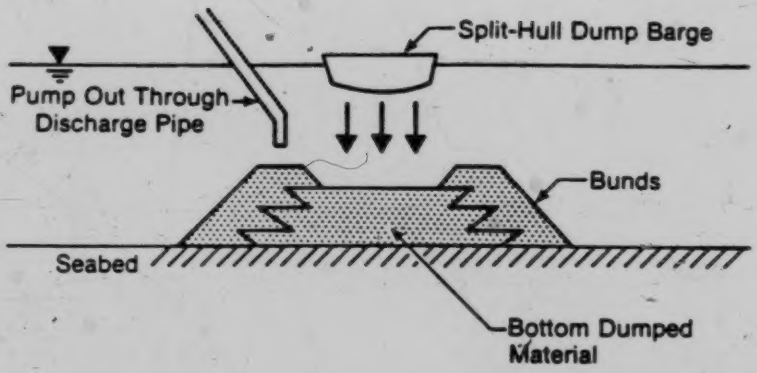
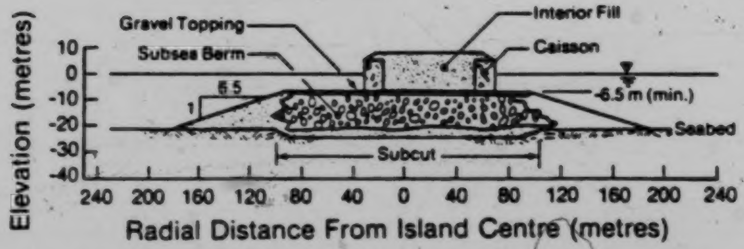
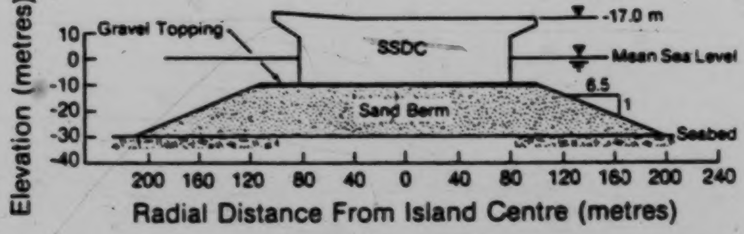

Split-Hull Dump Barge
Pump Out Through
Discharge Pipe
Bunds
Seabed
Bottom Dumped
Material

FIGURE 2.7 - Construction method for Tarsiut Island


Gravel Topping
Interior Fill
Subsea Berm
Caisson
6.5
1
-6.5 m (min.)
Seabed
Subcut
10
0
-10
-20
-30
-40
Elevation (metres)
240
200
160
120
80
40
0
40
80
120
160
200
240
Radial Distance From Island Centre (metres)

(a) Tarsiut N-44 Multiple Concrete Caisson System


-17.0 m
Gravel Topping
SSDC
Mean Sea Level
6.5
1
Sand Berm
Seabed
10
0
-10
-20
-30
-40
Elevation (metres)
200
160
120
80
40
0
40
80
120
160
200
240
Radial Distance From Island Centre (metres)

(b) Uviluk P-66 Single Steel Drilling Caisson System

FIGURE 2.8 - Cross section of hybrid exploration structures (modified after Mitchell, 1984)

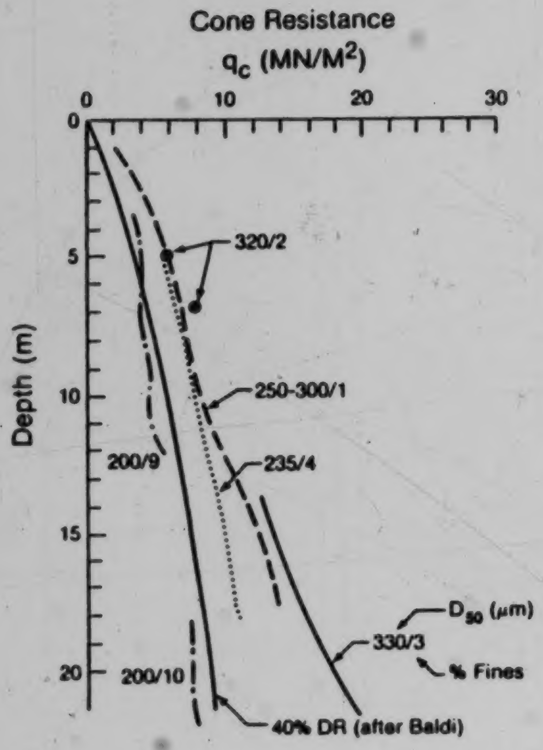


FIG. 2.9 - Cone resistance as a function of D_{50} and amount of fines (modified after Berzins and Hewitt, 1984)

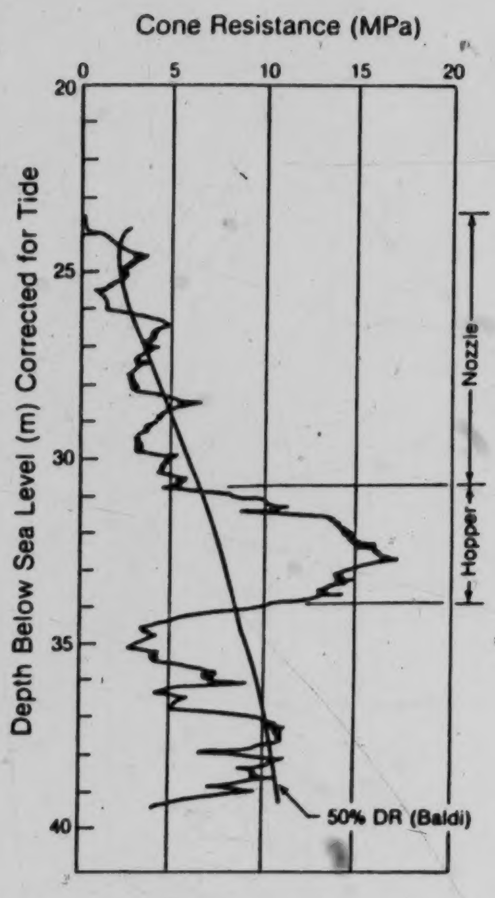


FIG. 2.10 - Nozzle and hopper density comparison at Nerlerk (modified after Mitchell, 1984)

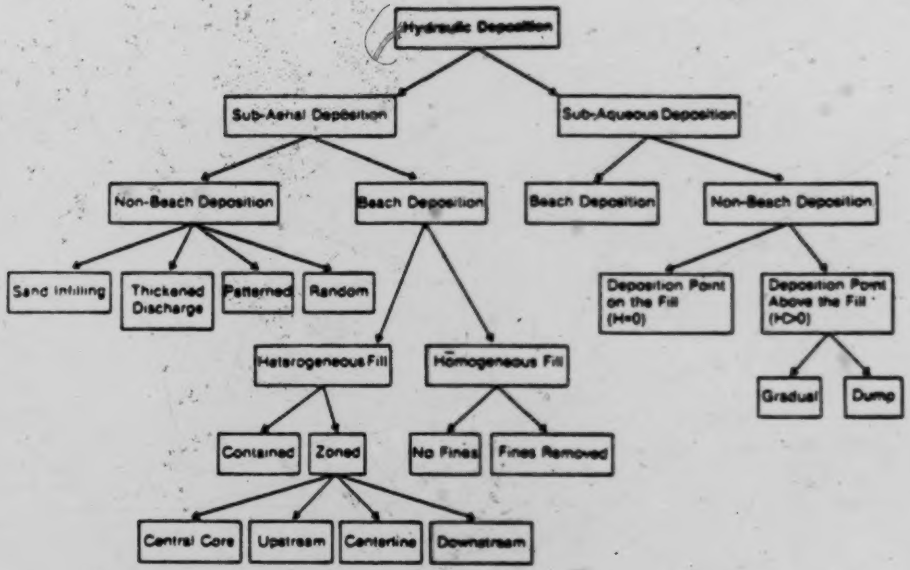

Hydraulic Deposition
Sub-Aerial Deposition
Sub-Aqueous Deposition
Non-Beach Deposition
Beach Deposition
Beach Deposition
Non-Beach Deposition
Sand Infilling
Thickened Discharge
Patterned
Random
Deposition Point on the Fill (H=0)
Deposition Point Above the Fill (H>0)
Heterogeneous Fill
Homogeneous Fill
Gradual
Dump
Contained
Zoned
No Fines
Fines Removed
Central Core
Upstream
Centerline
Downstream

FIGURE 2.11 - Classification of hydraulic deposition

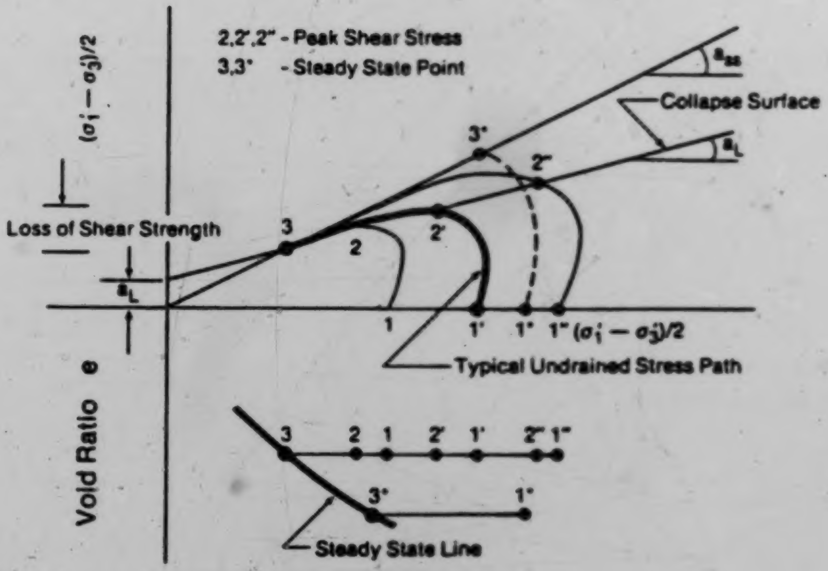

$(\sigma_1' - \sigma_3')/2$
2,2',2" - Peak Shear Stress
3,3' - Steady State Point
a_{ss}
Collapse Surface
a_L
Loss of Shear Strength
a_L
3
2
2'
3'
2"
1
1'
1'
1"
$(\sigma_1' - \sigma_3')/2$
Typical Undrained Stress Path
Void Ratio e
3
2
1
2'
1'
2"
1"
3'
1'
Steady State Line

FIGURE 2.12 - The collapse surface (modified after Sladen et al., 1985)

Chapter 3

Laboratory Tests to Study Hydraulic Fill

3.1 - INTRODUCTION

There is a clear appeal to the use of laboratory tests in order to study hydraulic fills. In a laboratory flume, the mechanics of the deposition process can be observed closely and tests can be performed in a more economical and more controlled manner than at field scale. Hence it becomes possible to perform a larger number of tests and study the effects of several variables. A logical approach to laboratory tests would be to scale model a hydraulic fill using the concepts of hydraulic modelling.

Hydraulic modelling, or scale modelling, is based on the fact that the equation that describes a particular phenomenon is independent of scale if it is normalized in such a way that all the algebraic terms are dimensionless. If we build a system (model) that is geometrically similar to the one we want to study (prototype) and have characteristics such that the constants of the normalized equation (a set of dimensionless numbers) have the same values as in the prototype, then both systems are described by exactly the same equation. In addition, because of their geometrical similarity, both systems have identical boundary conditions and the solution of the normalized equation is the same for both systems. Therefore we can study the model (small scale) and transfer the information obtained to the prototype. Although the concept of scale modelling has been described through the use of an equation, the knowledge of the equation that describes the physical phenomenon is not essential to scale modelling, since it is not being solved directly. It is necessary and sufficient to know the set of parameters that characterize the phenomenon (Yalin, 1971), which allows the immediate determination of a set of dimensionless variables and consequently establishes the criteria of dynamic similarity. Actually, one of the most important advantages of this method is that it can be developed directly from the parameters and does not require an equation relating them. This extends its applicability to phenomena that still cannot be described by an equation. Yalin (1971) even points out that it is not only simpler but also safer to use the parameters themselves rather than the relationship. Scale modelling is a very common approach in the study of hydraulic systems with non-erodible boundaries, where for turbulent flows of a viscous fluid the dimensionless variables that characterize the flow are the Reynolds number and the Froude number.

However, there are difficulties scaling down hydraulic phenomena that involve sediments. One limiting factor is that the sediment size can only be scaled down to a certain point where it becomes so fine that cohesive forces are introduced, invalidating the model. Moreover, grain shape and grain surface characteristics affect resistance to flow, sediment transport, equilibrium slope, deposit density, etc., and are very difficult if not impossible to scale. There are also practical limits to varying parameters such as fluid density, fluid viscosity and sediment density.

Southard et al. (1980) showed that the transport of loose sediment by steady unidirectional flows can be successfully modelled by a Reynolds-Froude model. Two flume runs were performed at different geometric scales, both using water and quartz sand. A scale ratio of 1.66 was obtained by using different sediment sizes and different water temperatures. The width of the flume was also changed to keep the width/depth ratio constant. The results of the scaled hot water run basically coincided with the results of the cold water run. However, this is practical only for small scale ratios (up to 2.5), which restricts its applicability. Also, it is not clear yet whether the same is valid for slurry deposition, since Southard et al.(1980) tested the effect of a flow of water on the movement of loose sediment that was already placed on the flume bottom.

An alternative approach to circumvent the difficulties associated with standard scale modelling techniques is to consider flume tests as fundamental tests, small systems in their own right, and not a scaled version of any prototype. Fundamental tests can help the understanding of the physical phenomena and the development of basic concepts and formulations that can be used to study the real scale situation at least in a qualitative way. In this case, whether numerical values can be transferred directly from the laboratory to the field becomes a secondary issue. Good examples of the utilization of this approach are the studies on alluvial fans performed by Hooke (1967) and Weaver (1984), and the flume tests carried out by the U.S. Geological Survey (Simons and Richardson, 1963; Guy et al., 1966). These flume tests led to the development of the concepts of bedform and bedform phase diagram that provide a fundamental basis for understanding several aspects of alluvial sedimentology, even though the actual results of these tests may be only qualitatively applicable in most field situations.

Although viewed with reservation by many geotechnical specialists involved with the design of hydraulic fills, flume deposition tests are routine in other areas such as sedimentology (especially as fundamental tests) and hydraulic engineering.

In this chapter, flume tests carried out to study hydraulic fills are described and a summary of the results is presented.

3.2 - PREVIOUS EXPERIENCE WITH FLUME DEPOSITION TESTS

The literature contains a profusion of flume tests involving flow of water and sediment. Most of these tests were carried out to study resistance to flow, sediment transport rates, bedform and stratigraphy of the deposit for specific flow conditions and a particular sediment within the context of sedimentology or hydraulic engineering. Only a few of the flume tests presented in the literature were performed specifically to study hydraulic fills Ferreira et al., 1980; Blight et al., 1985; Boldt, 1988; de Groot et al., 1988; Fan, 1989; Winterwerp et al.,1990). The emphasis in most of these cases differs from the hydraulic/sedimentologic studies as does the experimental procedure.

The objectives of the flume tests for hydraulic fill studies were usually the determination of the equilibrium slope for a particular set of deposition conditions and/or the study of the physico-mechanical properties of the deposited material. The basic procedure commonly adopted consists of preparing a slurry with a pre-established proportion of sediment to water and of depositing it at specified flow rates. In most cases the slurry is prepared in a mixing tank before being delivered to the flume. This tank requires an agitation system to ensure that a slurry of constant concentration will be delivered throughout the test. In some other cases, sand and water were discharged independently forming the slurry on the way to the flume (Ferreira et al.,1980) or sand was discharged directly in a flume with running water (Fan,1989). Typical data from flume tests to study hydraulic fills are the geometry and the deposit characteristics, which are determined after the deposition stops and the flume is drained. Results of the flume tests reported in the literature are compared to the results obtained in this study and presented in the next chapter.

3.3 - DESIGN OF THE EXPERIMENTAL STUDY

Due to the difficulties associated with the application of formal scale modelling procedures to study the formation of a fill by slurry deposition, the approach proposed by Hooke (1968) was adopted to design the laboratory tests. Hooke (1968) proposed an informal criterion of similarity, "similarity of process", in which the laboratory systems are viewed as small systems (not scale models of prototypes) and their performance is made similar in terms of process to the general systems being studied.

Basic requirements for using the similarity-of-process approach are that (Hooke,1968, p.392):

a) gross scaling relationships are met.

b) the model reproduces some morphologic characteristics of the systems being investigated.

c) the processes which produced a certain characteristic in the laboratory system can be logically assumed to have the same type of effect on natural systems.

The similarity-of-process approach is a straightforward approach that provides a basis for hypothesis generation and that may reveal new information regarding processes and trends. However, it precludes the direct extrapolation of results from laboratory to field, although qualitative conclusions drawn from the experimental system are still applicable to field scale systems. This kind of approach seems to be better suited to the study of the mechanics of hydraulic deposition in which the interest is not in modelling a particular fill, but instead in investigating general concepts relevant to all hydraulic fills.

In the design of the laboratory equipment, the gross scaling relationships were met by keeping the Reynolds number well above turbulent limits since field flows are turbulent, and by keeping the Froude number within the range of estimated Froude numbers from field flows. Requirements *b* and *c* of the similarity-of-process approach listed above, had to be met by observing the performance of the experimental system and by making adjustments to the equipment and to the testing procedure. By the time the laboratory tests started, a first field test had already been carried out (see Chapter 5), so these adjustments could be made based on information acquired from the field as well as experience with other hydraulic fills.

The main objectives of the laboratory tests were to study the mechanics of hydraulic deposition and the effects of the composition of the slurry and placement method on the properties of the fill. The experimental study was designed to study specifically the influence of total flow rate, slurry concentration and grain size of sand on the geometry, grain size distribution and density of the fill. The fabric of the deposited material was also studied and the main results are presented in Chapter 6.

3.4 - TEST PROGRAM

3.4.1-Equipment and test procedure

The laboratory equipment consisted of a 6.1 m long and 0.60 m wide flume and its feeding and drainage systems (Photos 3.1 and 3.2). The flume had clear Plexiglass walls and was divided lengthwise by a removable Plexiglass wall. This dividing wall allows one to run a test in one side of the flume while the material deposited on the other side is draining or is being sampled or prepared for the next test. It also allows for variation of the width of the flume being used, by moving the dividing wall sideways or by removing it.

The slurry is formed by feeding dry sand to a water stream. The water comes from a constant head reservoir and is controlled by an on-line flowmeter (*1*, Photo 3.2). The sand is fed at constant rate by a Vibra-Screw SCR-20 feeder (*2*, Photos 3.1 and 3.2) that consists of a hopper and a rotating auger, each one associated with an independent vibration system. Feeding rates are adjustable and once set, remain fairly constant even after several hours of continuous operation. Sand and water mix by turbulence on a chute (*3*) on the way down to the discharge point forming the slurry. Independent water and sand feeding systems have the advantage of being simple and providing slurry of constant composition for an indefinite period of time. This method also avoids the drawbacks associated with mixing tanks and allows easy adjustment of the flow rate of water or sand both prior to and during a test.

The slurry is discharged onto the fill by a device that spreads the flow uniformly across the width of the flume (*4*). Thin metal vanes inside the spreader assure an even distribution of the flow and direct the flow lines parallel to the flume walls. The spreader was designed to create a one dimensional discharge and to minimize the effects of the walls by having the flow parallel to a hydraulically smooth wall. Observation of the operation of the spreader indicates it was successful in this matter. The flow spreader was connected to the chute (*3*) where the slurry was formed by a funnel (*5*) and a flexible corrugated hose (*6*) that allowed free vertical movement of the spreader. The flow spreader was hung up on the shaft of a variable speed electric motor (*7*) placed above the flume. A floater (*8*) and a switch automatically controlled the motor in order to keep the distance between the spreader and the rising sand fill constant and equal to a predetermined value during the whole test. At the downstream end of the flume, drainage is provided by a constant level drain (standpipe) with adjustable height. A buffer causes the head loss necessary to settle most of the solids still in suspension, simulating the hydraulic fill's pond.

The tests were carried out by depositing slurry at a certain concentration and flow rate onto a smooth bed of pre-deposited sand. If this initial sand surface was too flat for the imposed flow conditions, preferred deposition at the upstream side would steepen the profile. However, if the initial slope was too steep, the flow would erode on the upstream side and deposit on the downstream side until the overall slope is flattened to its equilibrium value. The sand profile was determined at several time intervals during the test. The deposition proceeds until the equilibrium slope is attained (approximately parallel sand profiles) and there is enough material deposited in the flume to allow for undisturbed sampling, i.e., the deposit is thicker than 25 cm close to the discharge point and at least 10 cm thick further downstream.

Six to twelve undisturbed samples were taken along the flume and were used for density determination, triaxial tests and fabric study. Six to nine remolded samples were taken for grain size distribution analysis. Undisturbed samples were taken by statically pushing sampling tubes into the sand bed. Two sizes of sampling tubes were utilized, with diameters 7.5 and 10.0 cm. In order to study the disturbance caused by this sampling method, half a sampling tube was pushed into the sand bed contiguously to the flume transparent wall (Photo 3.3). By analyzing the bending of the thin layers of sand, it was conservatively assumed that the outer 1 cm of the sample in contact with the sampler was disturbed. After the sampling tube was pushed in, the sample was slowly frozen from bottom up (a few samples that indicated heave after being frozen were discarded). The use of metal samplers induced more frost heave, while better results were obtained with thin-walled plastic samplers. After being completely frozen, the samples were removed from the sampler and the outer ring of supposedly disturbed material was trimmed off in a walk-in freezer (T = -25° C). The samples were kept frozen until required for testing.

Appendix A presents a detailed description of the laboratory equipment and test procedure.

3.4.2-Test program

Three different sands were used for the flume tests:

a) SS sand: commercially available as SIL-7 from Silsilica, Edmonton.

b) TS sand: tailings sand scraped from Syncrude's tailings dam beach in Fort McMurray, Alberta. Since the sand was obtained mainly from the upper part of the beach, it does not contain as much fines as the material being discharged at the dam. The drying process that this material was submitted to also contributed to the loss of fines.

c) KS sand: blasting sand marketed by Kiel Industries Ltd, Edmonton as sand #7.

All of these sands have mainly subangular quartz grains. The grain size distribution of each sand is presented in Table 3.1 and Figure 3.1. For each of these sands, tests were carried out using different flow rates (Q_t) and different slurry concentrations (C_w).

Slurry concentration has been defined in the literature in several ways and it is not always clear which definition is adopted. This can sometimes be confusing when results are to be compared. In the following paragraphs an attempt is made to clarify this point.

Slurry concentration C_w can be defined in terms of weight as:

$$C_w = \frac{W_s}{W_t} \tag{3.1}$$

where W_s is the weight of solids in a certain total weight of slurry W_t. Slurry concentration defined in terms of volume (C_v) is:

$$C_v = \frac{V_s}{V_t} \tag{3.2}$$

where V_s is the volume of solids in a total volume of slurry V_t.

In the Soviet literature, the proportion of sand and water in a slurry is usually quantified using parameters not familiar to the Western literature. The parameters commonly used by the Soviets are:

Concentration:

$$C = \frac{W_s}{V_t} \tag{3.3}$$

Consistency A:

$$C_A = \frac{h_1}{h_2} x \ 100 \tag{3.4}$$

Consistency B:

$$C_B = \frac{W_s}{W_t - W_s} x \ 100 = \frac{W_s}{W_w} x \ 100 \tag{3.5}$$

where: h_1 is the height of sedimented solids in a container of parallel walls.

h_2 is the height of water above the sedimented material.

W_w is the weight of water in a certain weight of slurry W_t .

Concentration as defined in equation (3.3) is the parameter commonly used in the USSR to define the amount of solids in the decant water and it is usually expressed in g/litre. Consistencies A and B are used to quantify the proportion of solids and water in the slurry being deposited on the fill. Consistency A is a convenient parameter for quick field verifications. It is dependent on how long the sample has been sitting still in the sampling container, however this factor is not important for most Soviet fills, since the amount of fines is limited to 5% (hydraulic fills for water retention dams where borrow area material can be chosen). Consistency B is more accurate but has to be determined in the laboratory.

Slurry concentration defined in terms of weight by equation (3.1) will be adopted throughout this work. Slurry concentration was varied between 1.5 and 40% during the experimental program and total flow rate covered the range of 3 to 20 l/min. Tables 3.2 to 3.4 present a summary of the tests undertaken with sands SS, KS and TS respectively.

SS sand was the first sand used, with the main objectives of checking the equipment, developing an adequate test procedure and covering the bottom of the flume with free draining material to function as a pervious foundation for subsequent tests. The tests performed with this sand (SS tests, Table 3.2) used two different heights between the flow spreader and the fill. Also during these tests, the aperture of the mouth of the spreader (exit gap) was varied. For subsequent tests this gap was simply left open and the height between the spreader and the fill was kept equal to zero (spreader always touching the rising fill). All tests were performed continuously, except for tests SS3 and TS5 that had intermediate drainage periods.

3.5 - SUMMARY OF TESTS RESULTS

A summary of the overall results of the flume tests is presented here. Appendix B summarizes the results of individual tests.

3.5.1-Flow description

The characteristics of the flow and the sand deposit in the flume are described here in detail because a good understanding of the exact type of flow is absolutely essential to any subsequent theoretical (and even empirical) analysis that may follow. The depths of flow that commonly occur in the flume are too small and the sediment concentration too high for the studies on rivers and alluvial channels to be directly applicable. However, the concentrations are not high enough for the problem to be analyzed as a debris flow. Therefore, any approach to be taken in the analysis has to be based on a proper understanding of the flow characteristics.

Due to the use of the flow spreader as a discharge device, the flow mostly started in a well distributed manner across the flume section. After leaving the spreader, the flow tended to concentrate in one or more meandering channels. At higher flow rates, the flow established a braided pattern formed by several channels rather than a single channel, with small channels repeatedly separating and re-joining around islands and bars. The degree of braiding seems to depend on the total flow rate. At the highest flow rates, the flow tended to cover most of the fill surface, minimizing the influence of islands and bars. There was a continuous and dynamic process of channel migration with associated events of erosion and deposition, abandoning and re-taking of channels and instability of islands and bars. On a bend of a channel, there is more deposition on the inside of the curve causing the bar to grow towards the channel while the channel moves away, usually eroding another bar.

The small size of the flume and the transparent walls, associated with the lack of fines in the slurry (which keeps the carrier fluid clear), favoured the observation of the flow and the sediment transport mechanism. Sand grains seem to sediment almost instantly after the slurry is discharged and then move mainly as bedload by rolling and sliding along the slope. According to Bagnold (1973) this thin layer of moving grains just above the stationary bed that form the bedload, transfers momentum to the bed by solid-solid contacts. This mechanism may influence significantly the density of the deposited material or stationary bed. During the flume experiments it was observed that the smaller grains were mostly transported continuously, while the larger grains tended to move intermittently. The bedload zone was only a couple of millimeters thick for the TS sand, for instance. This agrees with the estimate given by Williams (1970) that the thickness of the bedload zone is approximately 8 times the average grain diameter. Williams (1970) also points out that this thickness seems to be independent of the flow depth and directly proportional to the rate of sediment transport. The tractional character of the sediment deposition on the flume tests becomes evident by the presence of current lineation as shown in Photo 3.4. Current lineations are linear features a few grain diameters high and aligned parallel to the flow direction. They are usually more visible on top of bars than on the channel floor. Photo 3.4 also shows the presence of riffles, especially where the shallow flow over bars meets channel flow at the downstream side of the bar.

On certain occasions, a large grain at rest at a particular location causes smaller grains and mainly other large grains to get trapped in this place, which in turn causes more grains to be trapped, forming what has been called a "lag deposit" (Leopold et al., 1964; Rachocki, 1981). This phenomenon was observed in several flume tests, being buried or washed away as the deposition proceeds. Lag deposits were usually coarser than the average grain size of the sand being deposited and also seemed to be softer and deposited in a "less organized" manner than the material in other areas of the flume. They can be considered responsible at least in some degree to anomalous values of density and grain size distribution parameters.

In areas where flow concentration occurs, the velocity tends to increase causing water waves in phase with the sand bed, exactly as happens in the field. These waves are called antidunes and cause remarkable remolding of the sand bed. Usually antidunes moved upstream and disturbed the bed to a depth between 2 and 5 cm (5 to 10 times the water depth). The upstream migration of the waves gives rise to faint low angle laminations. The resulting stratigraphy (Photo 3.5) was characterized by poorly defined planar laminations that were mainly dipping slightly upstream. Planar lamination is defined as more or less distinctly laminated sets that are approximately parallel to some overall depositional surface (Harms et al.,1982). Photo 3.5 shows a well developed cross-bedding stratification that is formed where the flow enters the pond at the downstream end of the flume. Smaller scale cross stratification also happens at the downstream end of bars. The same process occurs upstream of the discharge point (flow spreader) where water accumulates, causing the sand to be deposited very loosely and in a very steep angle. These slopes, formed when the flow enters deeper water, are very soft and slump readily (Photo 3.6).

3.5.2 - Geometry

The sand deposit in the flume typically had a smooth and slightly concave profile. Test profiles are presented on Figures 3.2 to 3.26. In order to facilitate comparing profiles of different tests, the position of the crest was considered to be always at ordinate 1 m (called nominal height).

The geometry of the profile was affected by the values of slurry concentration and flow rate. It was also affected by the average grain diameter. The effect of the slurry concentration is shown on Figures 3.2 to 3.11. Each of these plots presents tests that were carried out using the same sand and the same flow rate. Hence the only difference among the tests in each plot is the slurry concentration. As all of these figures show, the higher the slurry concentration, the steeper the resulting profile. Figures 3.12 to 3.22 plot together tests performed with the same sand and the same slurry concentration in order to display the effect of flow rate. It is shown that the higher the flow rate the flatter the profiles, other variables being constant. Note that with the flume width (w) constant, the total flow rate (Q_t) and the specific flow rate ($q = Q_t/w$) are equivalent.

When comparing different sands (Figures 3.23 to 3.26), the results show that the coarser sand forms a steeper slope than the finer sand for the same testing conditions, as expected.

In order to summarize the results, an overall slope was defined as the difference between the elevation of the crest and the elevation at the edge of the pond, or maximum height divided by the distance between the crest and the end of the beach (edge of the pond) and expressed as a percentage. The overall slope increases as the slurry concentration increases and decreases as the flow rate increases. Figures 3.27 and 3.28 show these trends for the TS sand tests and Figures 3.29 and 3.30 present the same plots for the KS sand. For the same value of slurry concentration and flow rate, the coarser the sand the steeper the slope as shown in Figures 3.31 and 3.32.

3.5.3 - Grain size distribution

Hydraulically deposited materials usually present a variation of mean grain size with distance along the flow, a phenomenon which is called hydraulic sorting. In general, coarser grains deposit first and finer grains ~~are~~ deposited further downstream.

For the flume tests, however, the mean grain diameter D_{50} increased in a downstream direction for almost all of the flume tests. The same trend was observed for D_{90} and for D_{10} (Figures 3.33 to 3.35). The values of D_{10} of all TS samples taken along the flume were larger than the value of D_{10} for the tailings sand being discharged on the TS tests (Figure 3.33a), showing that the finer fraction of the TS sand was washed out during the deposition process. The average value of D_{50} of TS flume samples was similar to the D_{50} of the original tailings sand (Figure 3.33b). The values of D_{90} in the flume were slightly higher (Figure 3.33c), however this can be merely the effect of the loss of the finer fraction on the relative proportion of the remaining fractions. Both KS and SS tests yielded values of D_{10} and D_{50} along the flume that were similar to the initial parameters (Figures 3.34 and 3.35). The value of D_{90} increased with distance for KS and SS tests, but kept below the value of D_{90} for the input sand for the great majority of the samples, possibly representing a concentration of larger grains in singular points in the flume such as in lag deposits and at the toe of the backslope.

As a result of the fines being washed out for the TS sand, the flume samples were more uniform, i.e., they had a lower coefficient of uniformity (C_U) than the material originally discharged (Figure 3.36a). Most of the KS flume samples were also more uniform than the input material (Figure 3.37a), but the SS flume samples mainly had a higher coefficient of uniformity (Figure 3.38a). For the TS sand, the increase in D_{90} was more pronounced than the increase in D_{10} causing the parameter D_{90}/D_{10} to also show an increase with distance (Figure 3.36b). The opposite trend could be observed for the KS and SS sands with D_{10} increasing more than D_{90} resulting in a trend to decrease D_{90}/D_{10} along the flume (Figures 3.37b and 3.38b).

Trends of the variation of the grain size parameters with distance for each test do not seem to have any correlation with the values of slurry concentration and flow rate utilized in the test.

3.5.4 - Density

The density of the hydraulic fill was determined at several locations along the flume. No consistent trend of density variation with distance was observed (see Appendix B). In approximately half of the TS tests, the density decreased slightly with distance despite the increase in mean grain size and the slight increase in the coefficient of uniformity (C_U). In one quarter of the tests, the density was approximately constant with distance and for the remaining quarter, it increased slightly. These differences in trend were not accentuated and did not bear any relationship with the values of flow rate and slurry concentration utilized. In most KS tests, the density increased slightly with distance which is consistent with the coarsening of the material towards the downstream, but the density was either constant or decreasing with distance for some 30% of the tests.

Due to the lack of a consistent trend of the density along the flume, an average density was adopted for each test. Therefore each density value presented here represents an average of 4 to 28 values measured along the flume. Appendix B presents all individual values of density plotted versus distance from the discharge point. Respective values of grain size distribution parameters are also shown.

Both slurry concentration and flow rate seem to affect the average density of the material deposited in the flume. Figures 3.39 and 3.40 show the variation of the density with concentration and flow rate respectively for the TS tests, while Figures 3.41 and 3.42 present the same results for the KS tests. The density tends to decrease for higher values of slurry concentration and increase for larger flow rates. The length of the error bars shown in these figures corresponds to the standard error calculated considering all the density values obtained for each test. The question of the accuracy of density determination is discussed in Appendix C.

Figures 3.43 and 3.44 show the variation of average density with slurry concentration and flow rate, respectively. It demonstrates that the density increases with average grain size for the same slurry concentration and flow rate. Therefore, for constant testing conditions, the coarser the sand the higher the density. However, it should be noted that all three sands tests have similar coefficient of uniformity ($C_U = D_{60}/D_{10}$) and similar D_{90}/D_{10} (Table 3.1).

The presence of islands and bars may also have affected the local values of density, since the material deposited at these locations seems to be coarser and looser than the material deposited in a channel. Lag deposits also seem to be distinct from the rest of the fill, and because of their formation mechanism the grains end up deposited in a looser form. Due to the dynamic character of the flow, channels keep changing place, bars, islands and lag deposits get buried or eroded, resulting in a stratigraphy formed by a random sequence of events which is difficult to be determined afterwards. Since each event will have a certain specific influence on the density value, some scatter in the density values should be expected beyond the normal experimental scatter. Moreover, density is affected by the grain size distribution parameters, which also vary along the flume and have a significant scatter.

3.5.5-Comments

Flow velocity seems to be also an important variable for the hydraulic deposition process. In the case of these flume tests however, the experimental set-up was such that velocity was a function of the total flow rate and therefore was not controlled independently. Further investigation may be necessary to assess the effect of the flow velocity on the geometry and density of the resulting fill.

Some of the SS flume tests were performed with the flow spreader set at 25 cm above the fill level during the whole tests. As a result a plunge pool formed where the slurry jet hit the fill being deposited. Although only three tests were carried out using a height between the spreader and the fill, some preliminary observations were made.The existence of a plunge pool did not seem to have influenced the geometry or density of the fill to a great extent, but affected the stratigraphy of the upper part of the fill. The first 0.5 m after the plunge pool (panel 2) did not develop a well defined lamination as in the cases where the spreader was kept at the fill level. Also, some of the coarser grains were trapped in the plunge pool. The effect of the formation of a plunge pool on the characteristics of the fill seems to be important enough to require further investigation.

Flume test KS19 was carried out with the objective of observing the effect of a sudden drop on the downstream water level, i.e., the pond level in a field situation. When an equilibrium slope was obtained and the deposit of this test was approximately 10 cm thick on average, the water level was lowered about 15 cm. This caused erosion of the slope that had been previously deposited. The erosion process started at the edge of the pond, concentrated in two channels and progressed upstream in a few minutes, until it stabilized before reaching the discharge point. After the channels stopped progressing upstream, the flow started re-working the deposit until the equilibrium slope for the current flow rate and slurry concentration was again reached.

3.6 - SUMMARY AND CONCLUSIONS

Laboratory flume tests were performed to study the physical phenomena related to the deposition of a sand slurry to form a hydraulic fill. The similarity-of-process approach proposed by Hooke (1968) was adopted to design the tests, consequently a direct extrapolation of quantitative results from laboratory to field may not be possible.

Slurry concentration and flow rate were varied independently to study their effects on the characteristics of the fill. Three different sands were used to evaluate the influence of the mean grain size.

All the tests developed a concave slope, that was steeper for larger slurry concentrations, smaller flow rates and larger mean grain diameter. The average fill density tends to decrease as slurry concentration increases and as flow rate decreases. These conclusions should be verified at a field scale as they may be of considerable importance for hydraulic fill engineering.

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Table 3.1 - Average parameters for the sands used in the flume tests

SAND	D_{50} (mm)	D_{10} (mm)	% fines	D_{90}/D_{10}	D_{60}/D_{10}
SS	0.536	0.348	0	3.16	1.70
TS	0.178	0.090	4	3.16	2.17
KS	0.466	0.251	0	3.98	2.18

OBS: $D_{50}(SS) / D_{50}(TS) = 3.0$

$D_{50}(KS) / D_{50}(TS) = 2.6$

Table 3.2 - SUMMARY OF SS FLUME TESTS

TEST No.	Cw nom (%)	Qt(v)nom (l/min)	H fall (cm)	Exit gap (mm)	Duration (hs.)	Comments
SS1	5.7	14.9	0	2.34	4.0	
SS2	3.5	14.8	0	2.34	13.7	
SS3	6	14.9	25	2.34	48.0	not continuous
SS4	3.5	14.8	25	2.34	-	not completed
SS5	6	14.9	25	4.57	5.7	

Table 3.3 - SUMMARY OF KS FLUME TESTS

TEST No.	Cw nom (%)	Cw aver (%)	Qt(v)nom (l/min)	Qt(v)fin (l/min)	Duration (hs.)	Comments
KS1	2	3.0	3	3.2	4.1	
KS2	6	-	5	-	22.3	not completed
KS3	6	6.4	5	4.3	12.2	
KS4	10	9.5	5	5.0	4.3	
KS5	15	14.5	5	4.5	4.7	
KS6	20	19.7	5	4.7	3.3	
KS7	25	23.9	5	4.7	3.2	
KS8	30	28.8	5	4.9	2.5	sand tongues
KS9	2	2.1	10	8.2	32.0	
KS10	6	6.5	10	8.3	7.5	
KS11	10	10.1	10	8.9	3.5	
KS12	15	13.3	10	9.9	1.9	
KS13	20	20.4	10	8.4	1.7	
KS14	10	9.0	20	20.9	2.1	
KS15	6	-	20	-	2.0	not completed
KS16	6	5.8	20	19.4	2.8	
KS17	2	1.9	20	18.9	9.8	
KS18	10	12.0	3	3.3	3.3	
KS19	10	13.3	5	4.2	2.8	water level drop

Table 3.4 - SUMMARY OF TS FLUME TESTS

TEST No.	Cw nom (%)	Cw aver (%)	Qt(v)nom (l/min)	Qt(v)fin (l/min)	Duration (hs.)	Comments
TS1	5.7	-	14.9	14.9	27.7	
TS2	10.7	-	12	12.0	3.0	
TS3	19.3	20.0	12	12.0	1.3	
TS4	19.3	-	12	12.0	1.5	
TS5	19.3	-	12	12.0	38.5	in 4 stages
TS6	2.0	-	5	5.0	21.0	
TS7	2.0	2.0	3	3.2	21.6	
TS8	6.0	7.1	5	4.8	7.9	
TS9	10.0	10.2	5	5.1	3.3	
TS10	15.0	17.1	5	4.6	3.7	
TS11	20.0	21.5	5	5.1	3.3	
TS12	25.0	26.2	5	5.0	2.9	
TS13	30.0	-	-	-	-	slurry did not flow
TS14	10.0	11.0	10	10.3	4.4	
TS15	2.0	1.8	10	8.5	12.6	
TS16	6.0	4.9	10	9.7	5.3	
TS17	10.0	10.2	10	10.1	3.3	
TS18	30.0	30.4	5	5.0	2.3	
TS19	35.0	33.9	5	4.9	3.0	
TS20	15.0	14.5	10	9.9	2.0	
TS21	20.0	19.7	10	10.0	2.3	
TS22	15.0	14.1	15	14.8	1.7	
TS23	2.0	-	20	-	11.0	
TS24	10.0	8.7	20	20.4	2.5	
TS25	6.0	5.3	20	19.8	1.9	
TS26	10.0	8.5	5	5.0	4.5	
TS27	2.0	1.5	20	20.8	7.4	
TS28	10.0	7.7	15	14.8	3.8	
TS29	6.0	4.5	15	14.9	5.2	
TS30	40.0	29.9	5	4.8	2.4	
TS31	25.0	18.0	5	4.8	3.0	
TS32	6.0	4.1	10	9.2	8.0	
TS33	2.0	3.1	3	3.2	28.6	
TS34	30.0	31.0	5	4.9	2.3	
TS35	35.0	35.4	5	5.1	1.2	
TS36	40.0	40.2	5	5.0	1.3	
TS37	2.0	2.5	3	3.2	28.2	

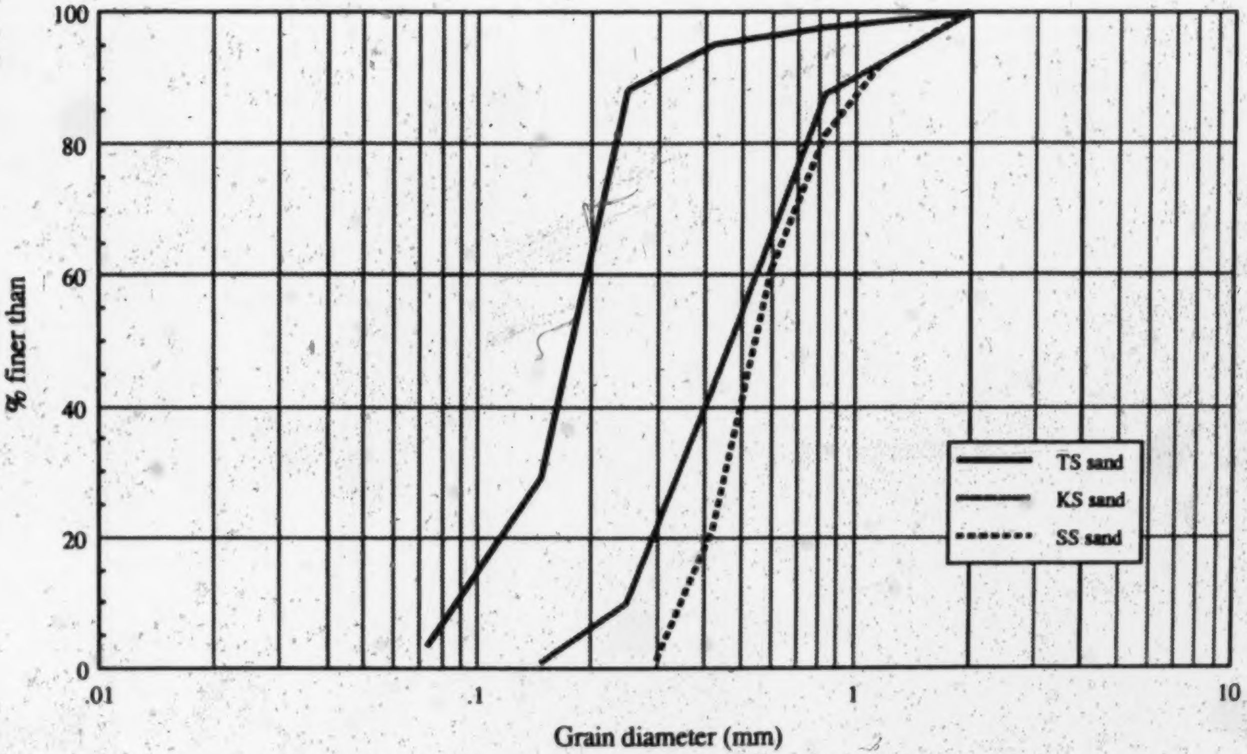


FIG. 3.1 - Average grain size distribution curves of TS, KS and SS sands used in the flume tests

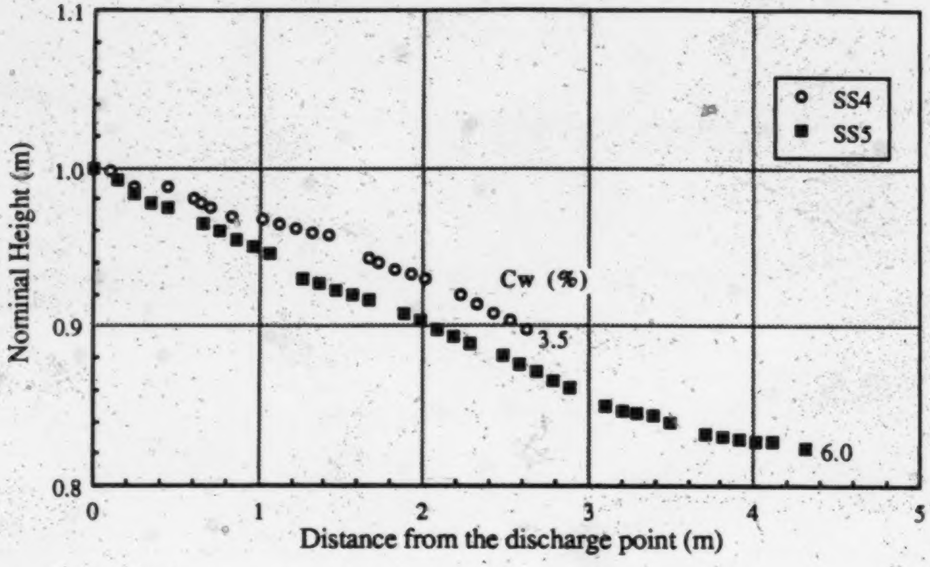


FIG. 3.2 - Profiles of SS tests with Qt = 15 l/min and various Cw

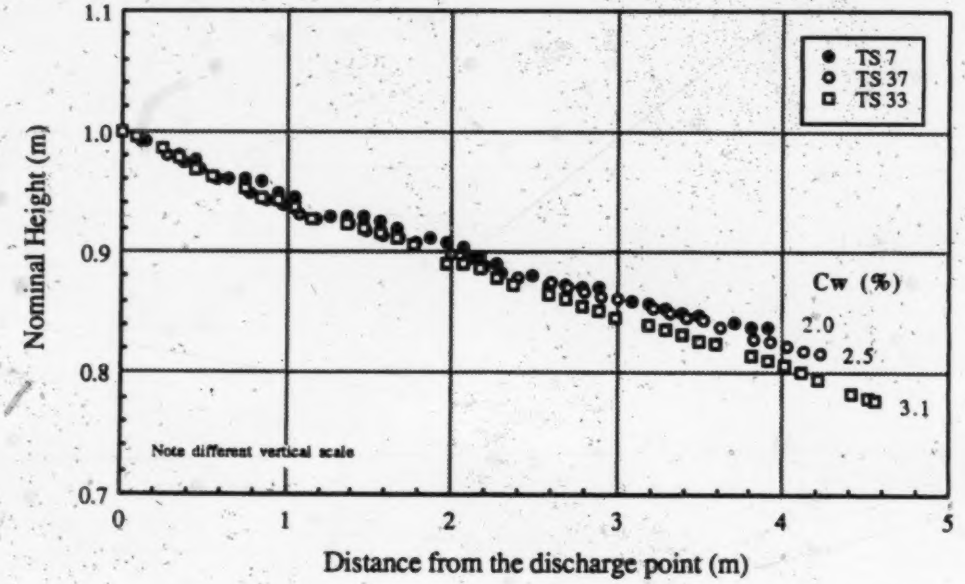


FIG. 3.3 - Profiles of TS tests with Qt = 3 l/min and various Cw

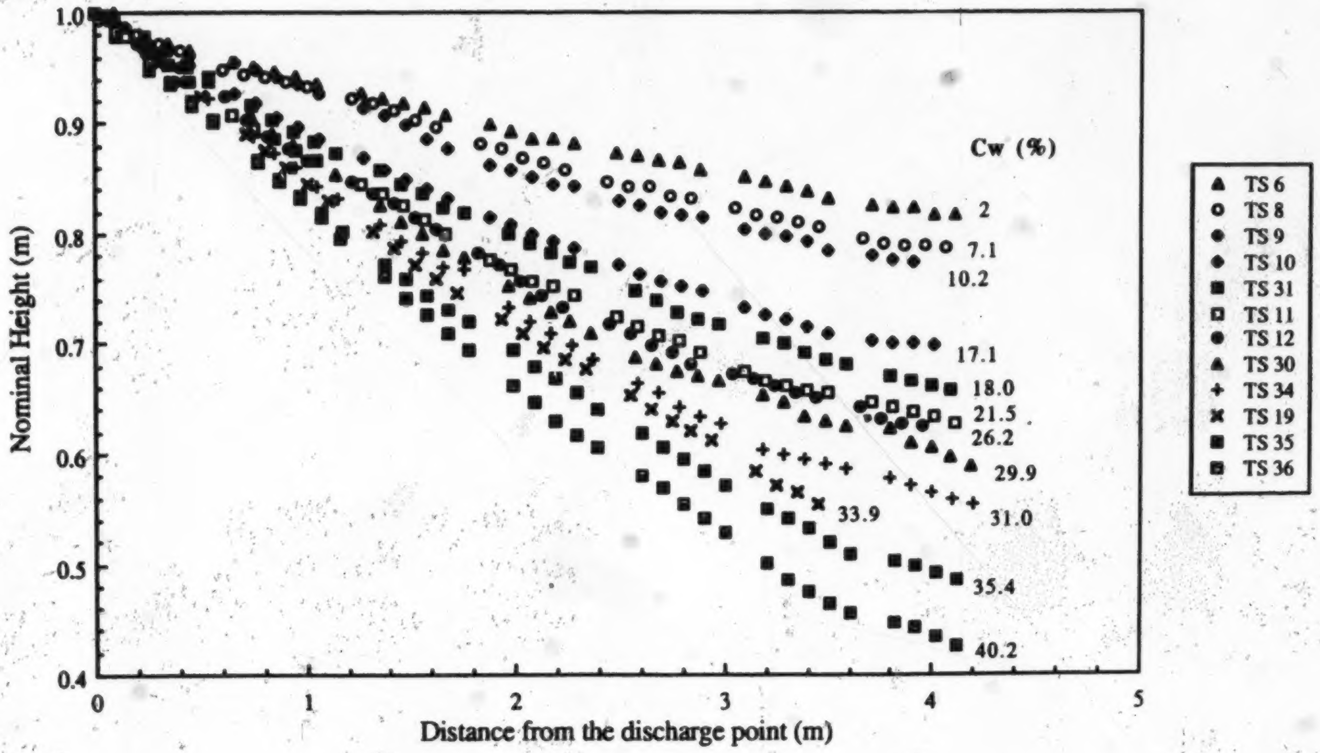


FIG. 3.4 - Profiles of TS tests with Qt = 5 l/min and various Cw

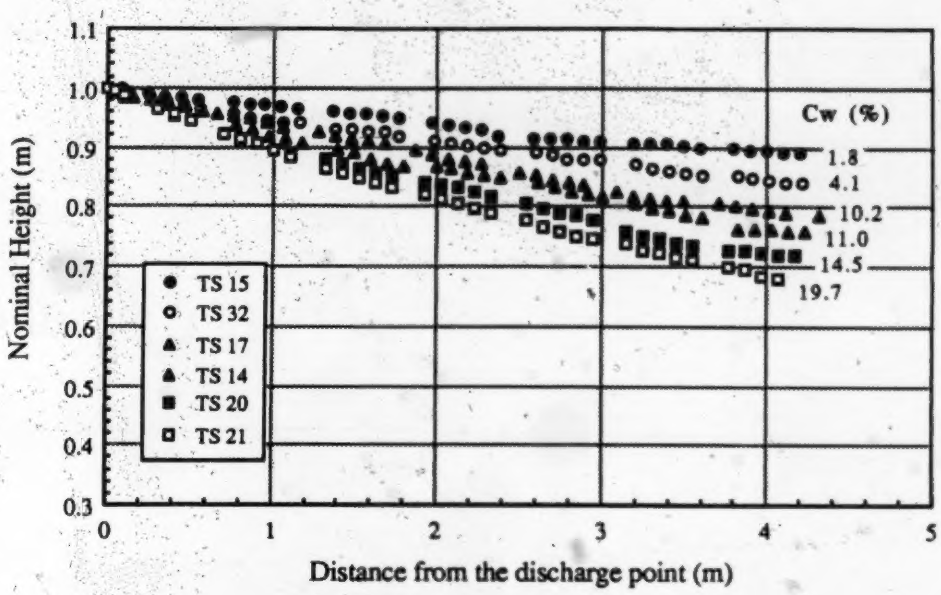


FIG. 3.5 - Profiles of TS tests with Qt = 10 l/min and various Cw

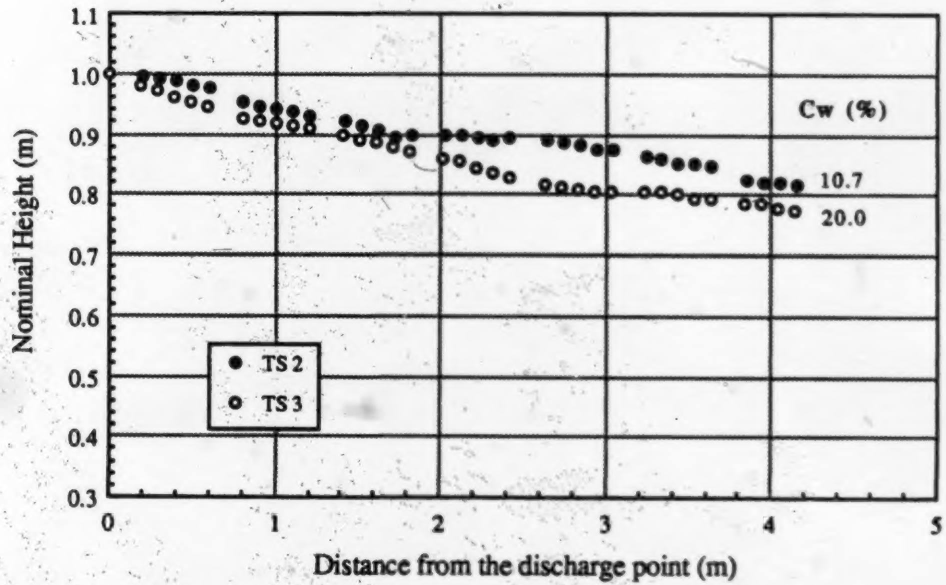


FIG. 3.6 - Profiles of TS tests with Q = 12 l/min and various Cw

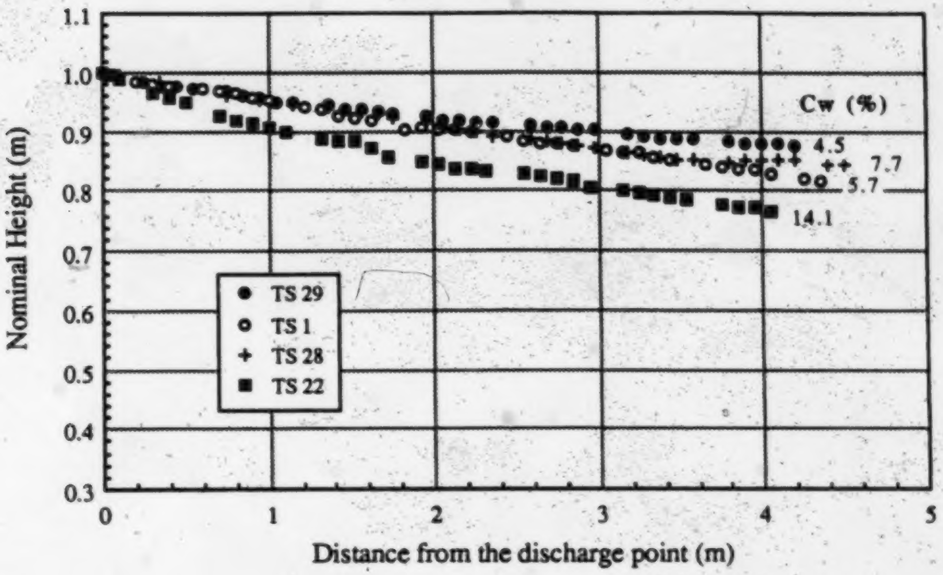


FIG. 3.7 - Profiles of TS tests with Qt = 15 l/min with various Cw

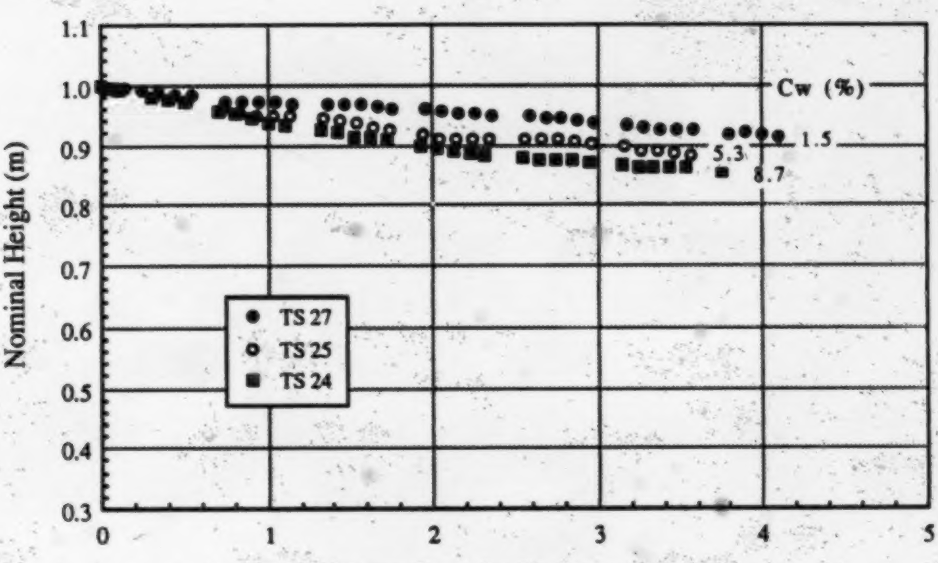


FIG. 3.8 - Profiles of TS tests with Qt = 20 l/min and various Cw

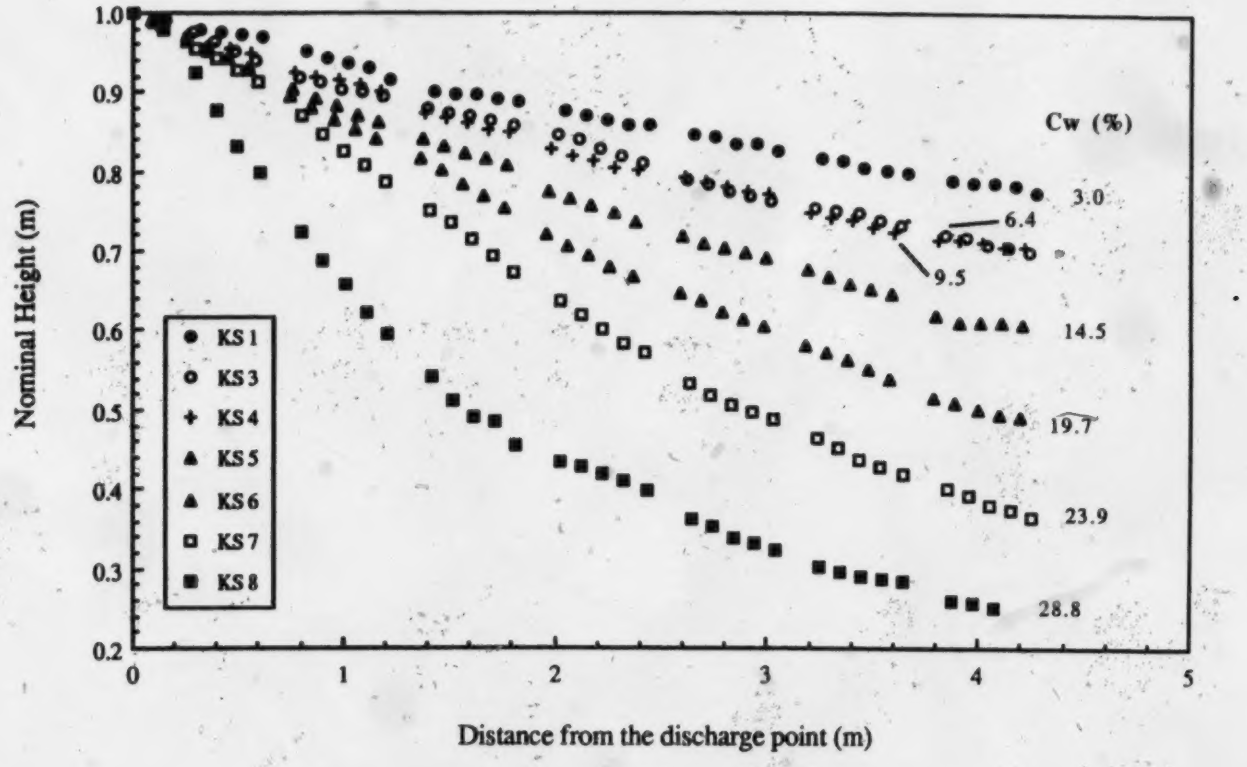


FIG. 3.9 - Profiles of KS tests with Qt = 5 l/min and various Cw

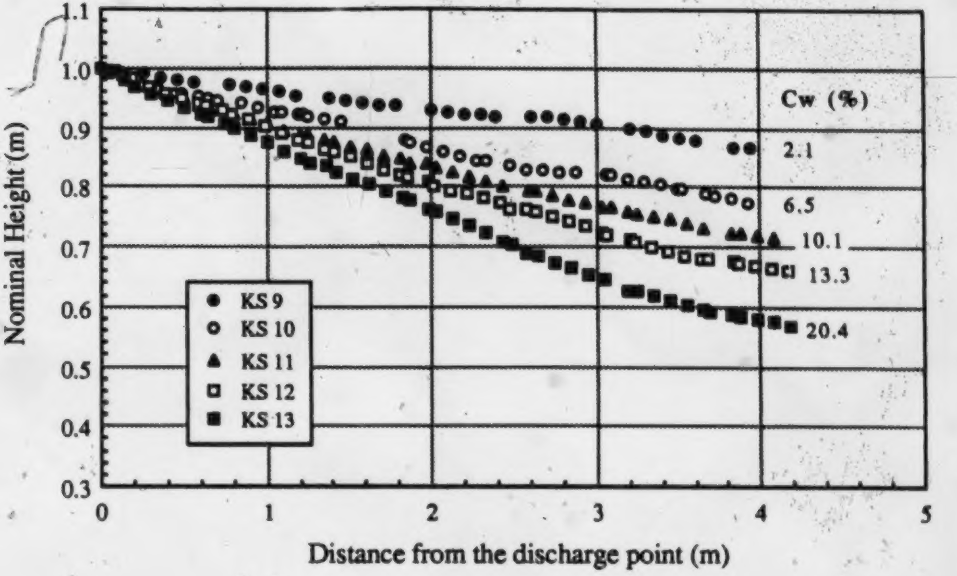


FIG. 3.10 - Profiles of KS tests with Qt = 10 l/min and various Cw

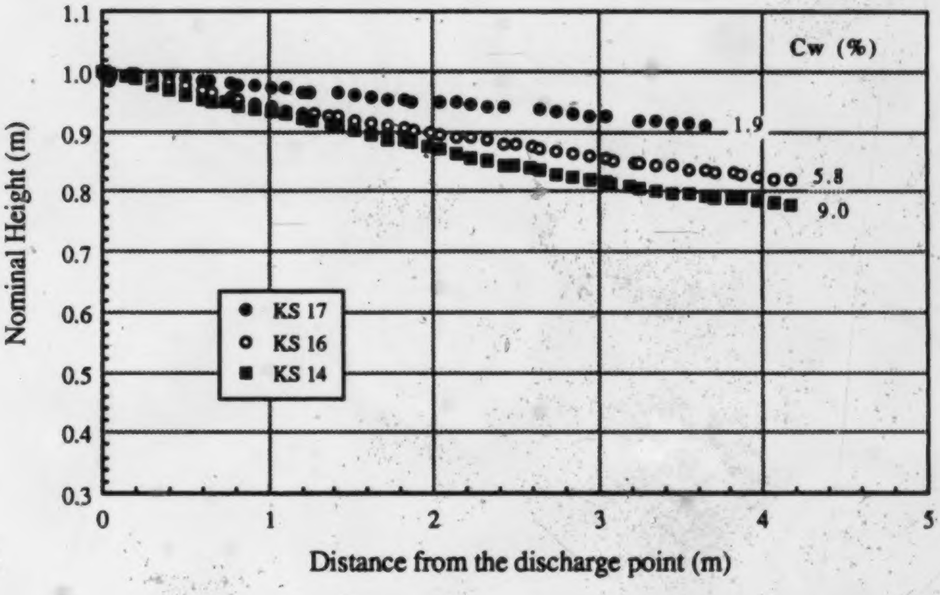


FIG. 3.11- Profiles of KS tests with Qt = 20 l/min and various Cw

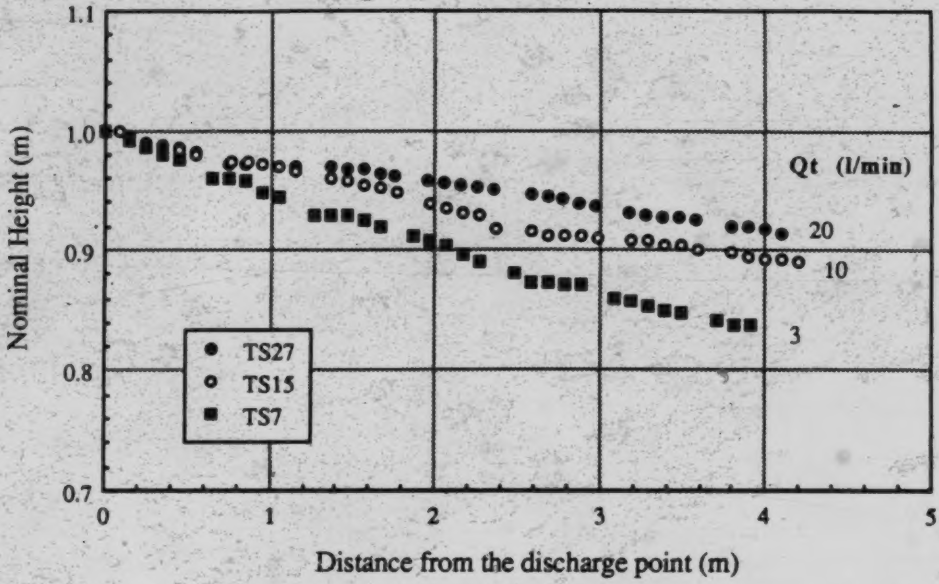


FIG. 3.12 - Profiles of TS tests with Cw = 2% and various Q

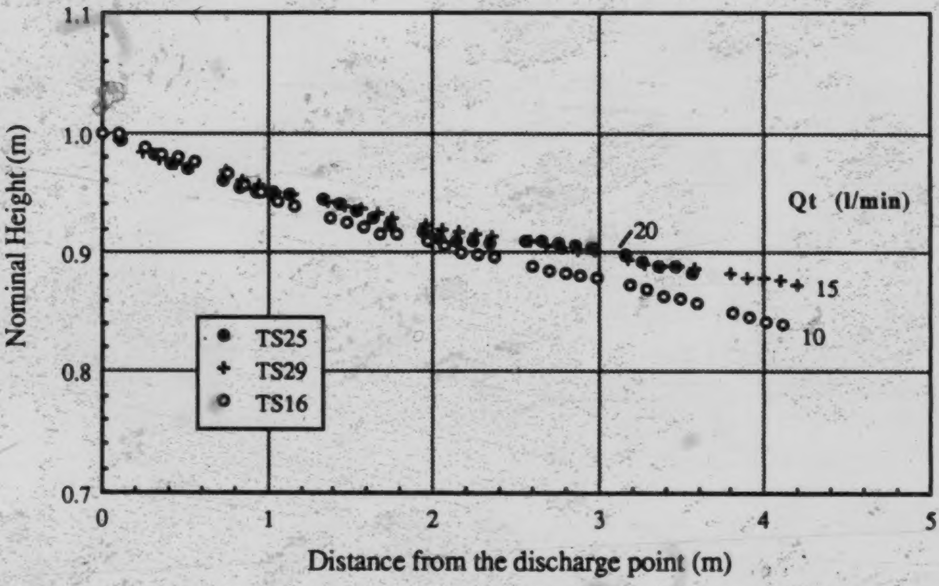


FIG. 3.13 - Profiles of TS tests with Cw = 5% and various Q

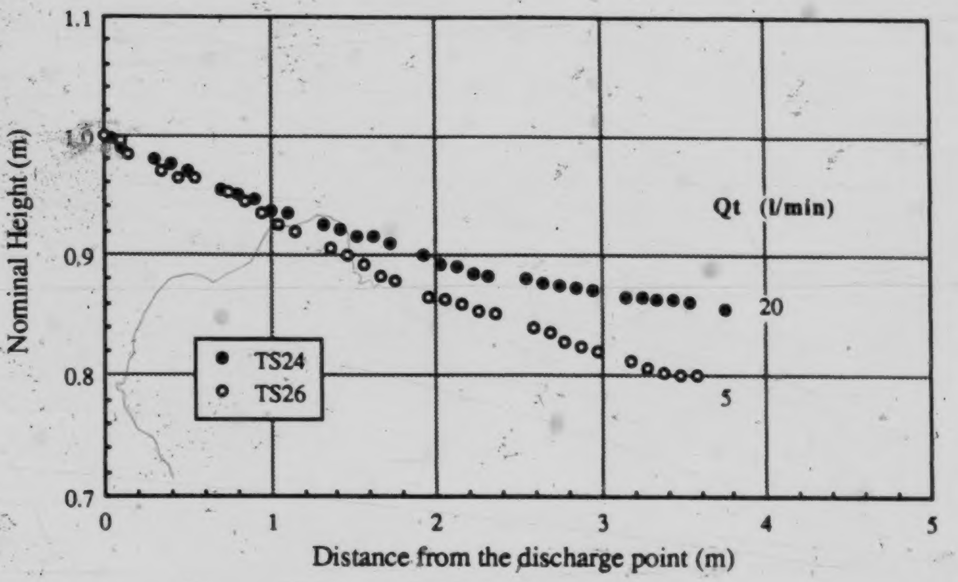


FIG. 3.14 - Profiles of TS tests with Cw = 8.5% and various Q

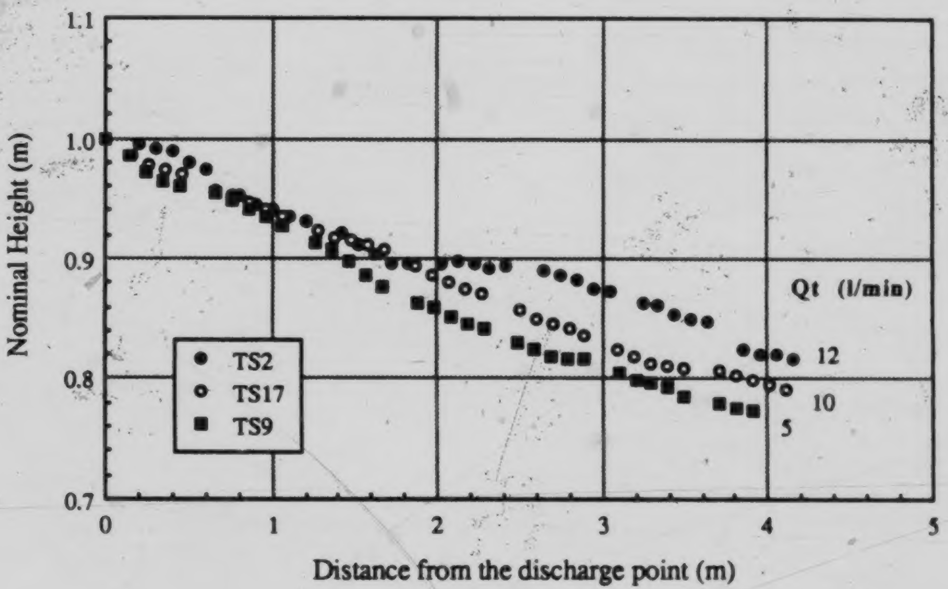


FIG. 3.15 - Profiles of TS tests with Cw = 10% and various Q

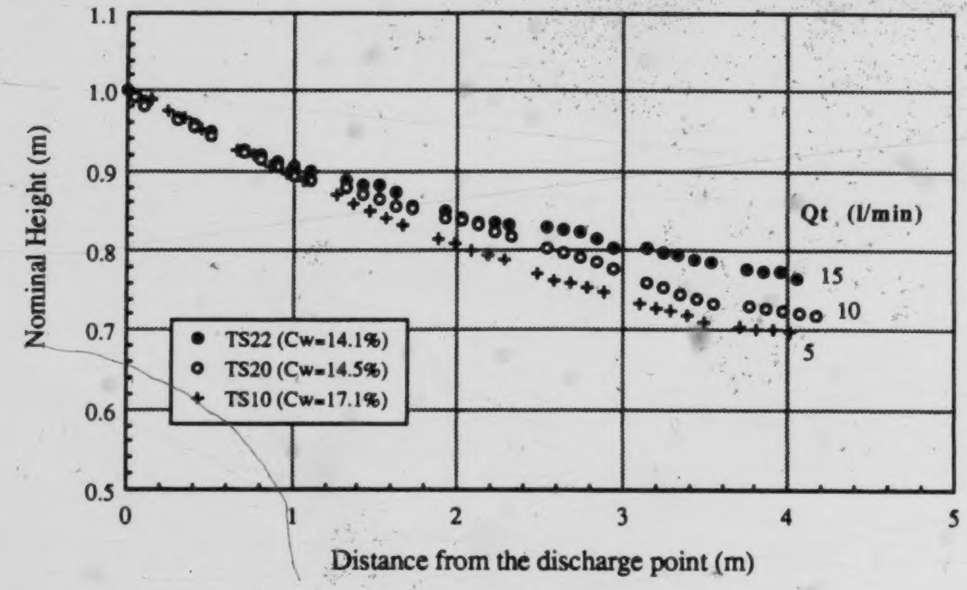


FIG. 3.16 - Profiles of TS tests with Cw ~ 15% and various Q

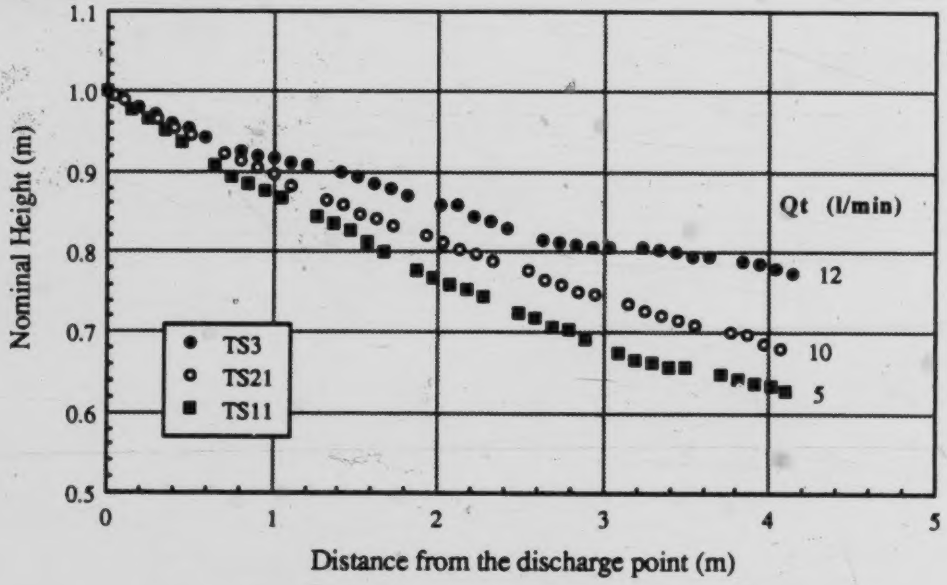


FIG. 3.17 - Profiles of TS tests with Cw = 20% and various Q

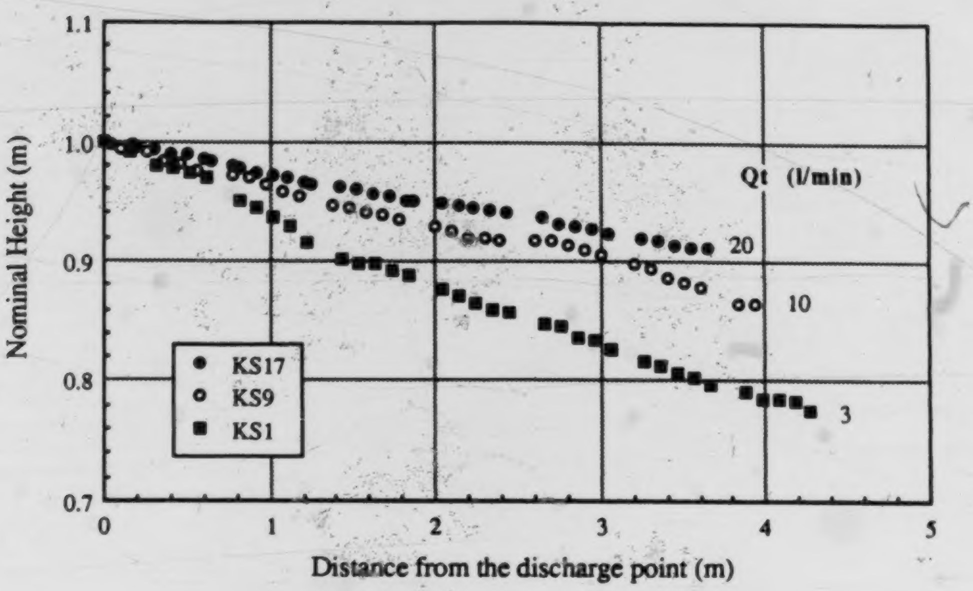


FIG. 3.18 - Profiles of KS tests with Cw = 2% and various Q

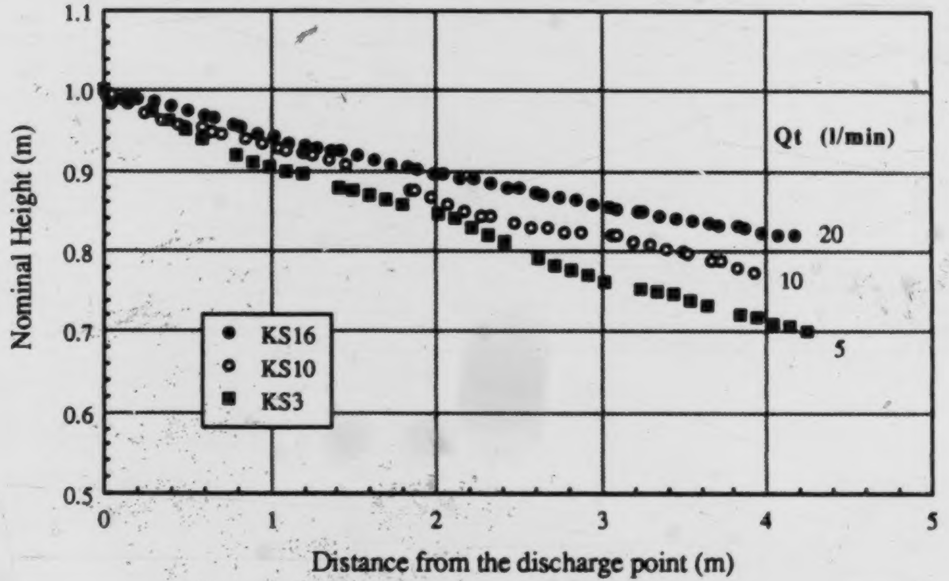


FIG. 3.19 - Profiles of KS tests with Cw = 6% and various Q

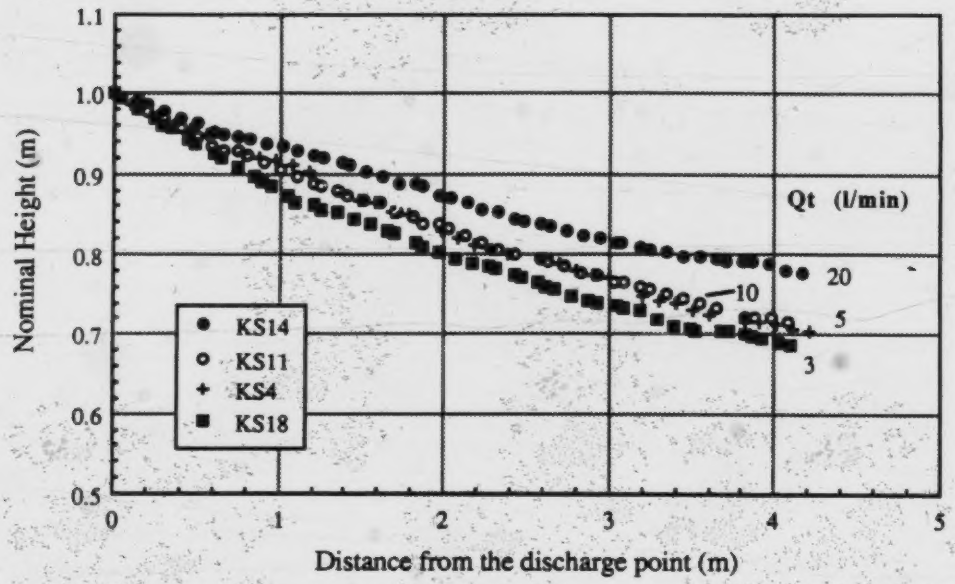


FIG. 3.20 - Profiles of KS tests with Cw = 10% and various Q

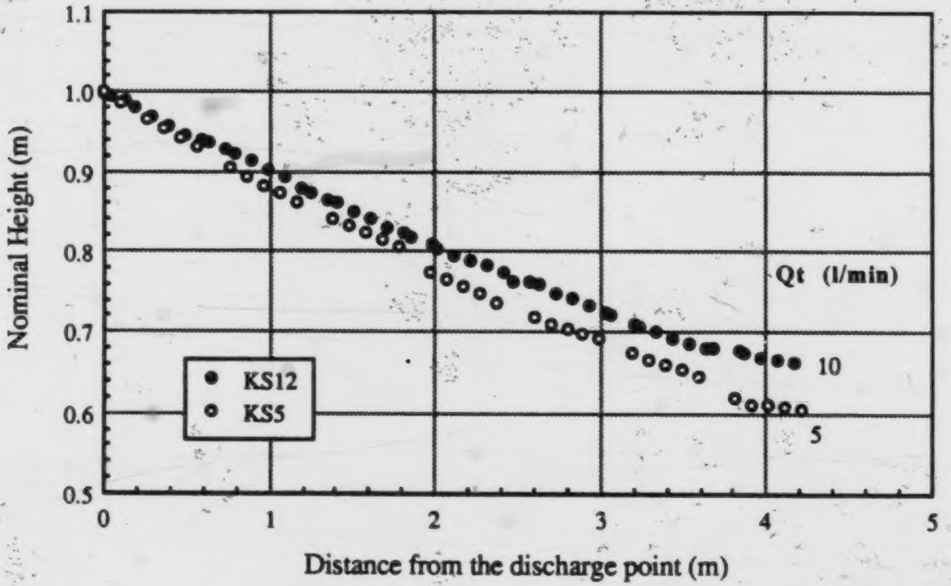


FIG. 3.21 - Profiles of KS tests with Cw = 15% and various Q

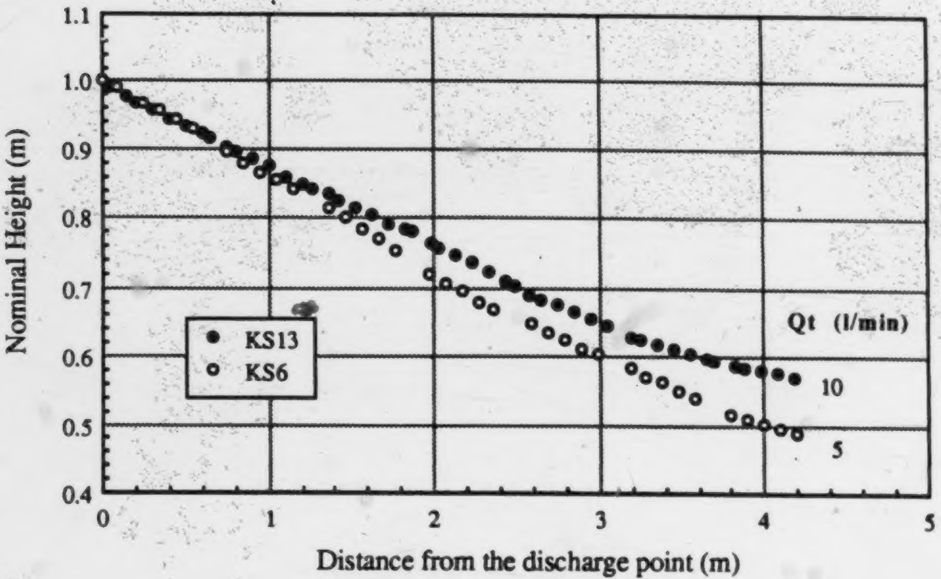


FIG. 3.22 - Profiles of KS test with Cw = 20% and various Q

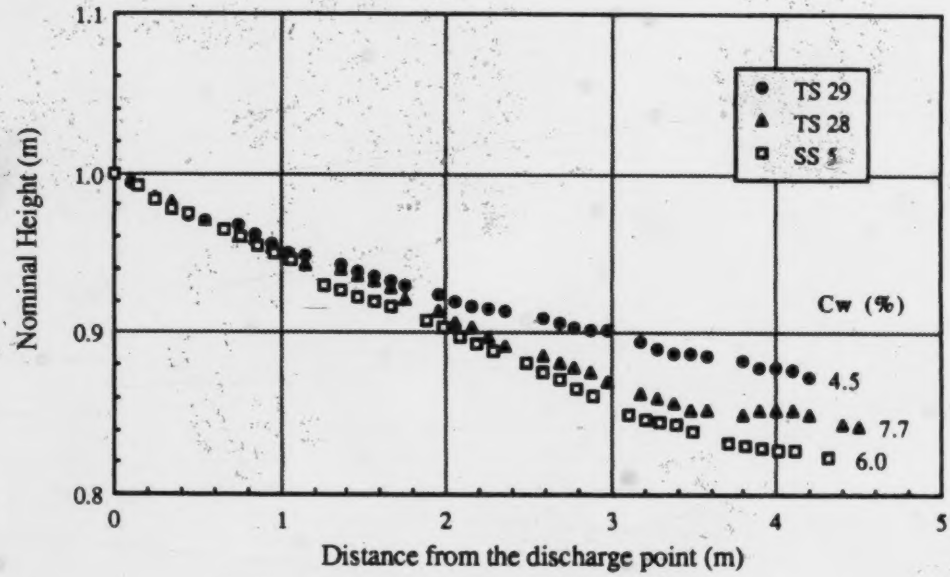


FIG. 3.23 - Comparison between SS and TS tests for Q = 15 l/min

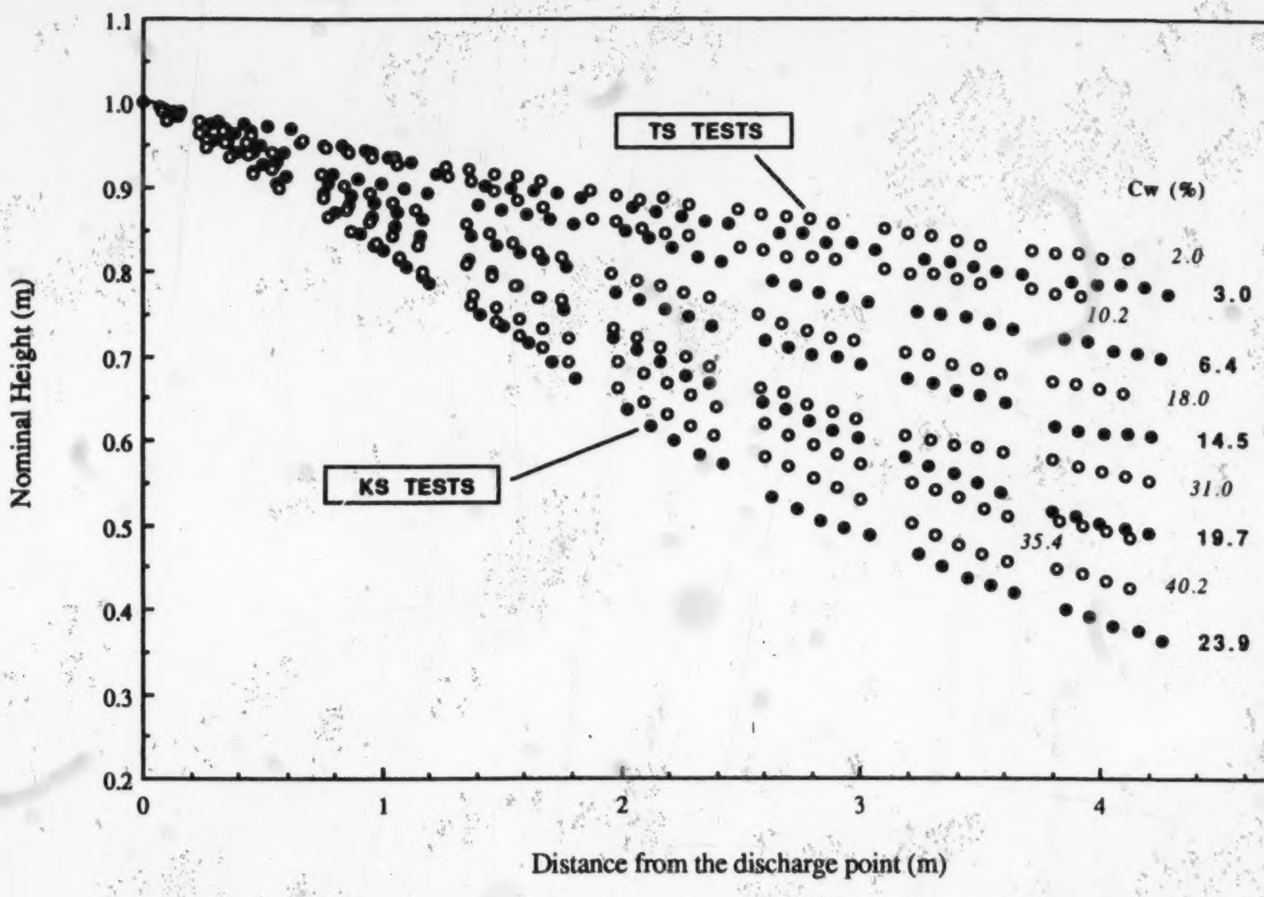


FIG. 3.24 - Comparison between KS and TS tests for Q = 5 l/min

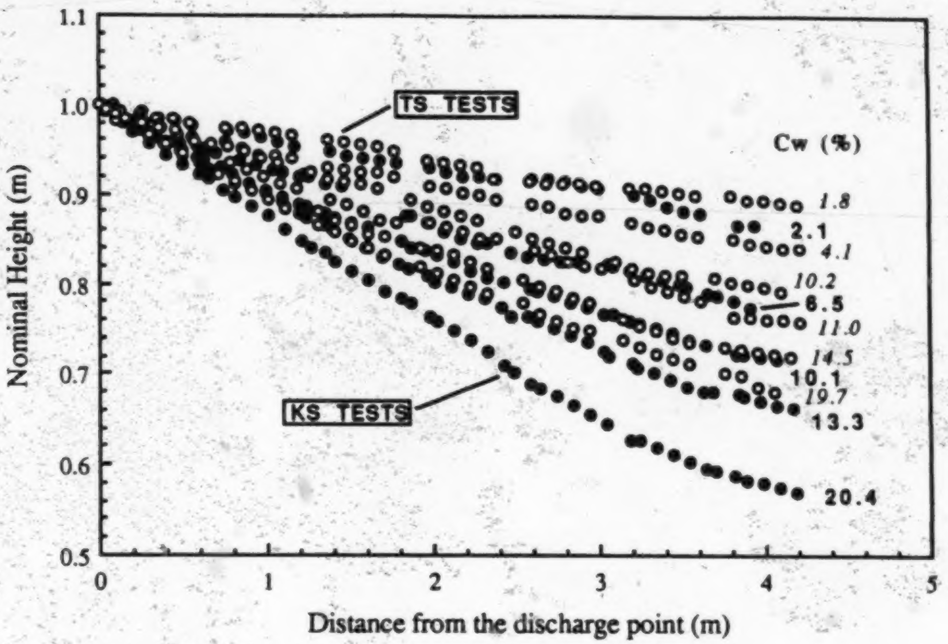


FIG. 3.25 - Comparison between KS and TS tests for Q = 10 l/min

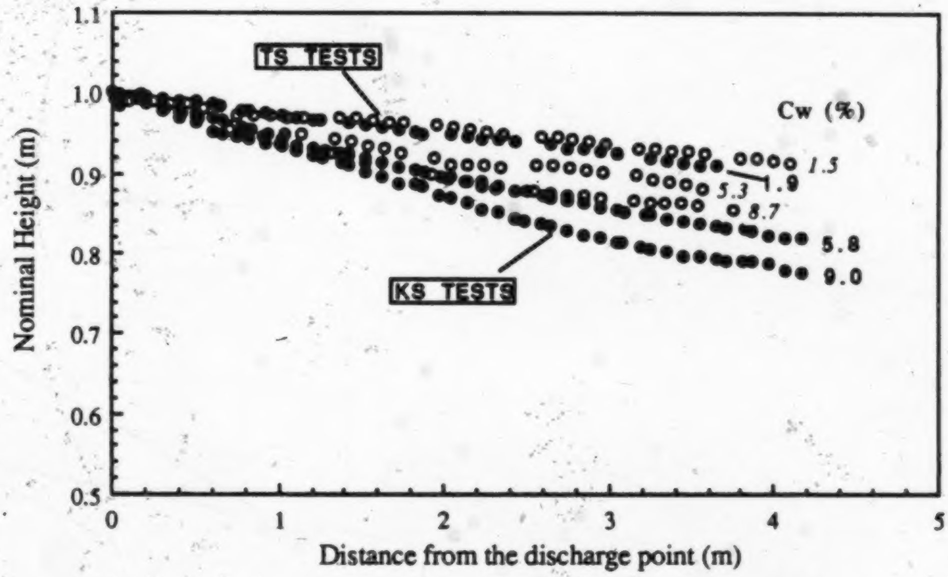


FIG. 3.26 - Comparison between KS and TS tests for Q = 20 l/min

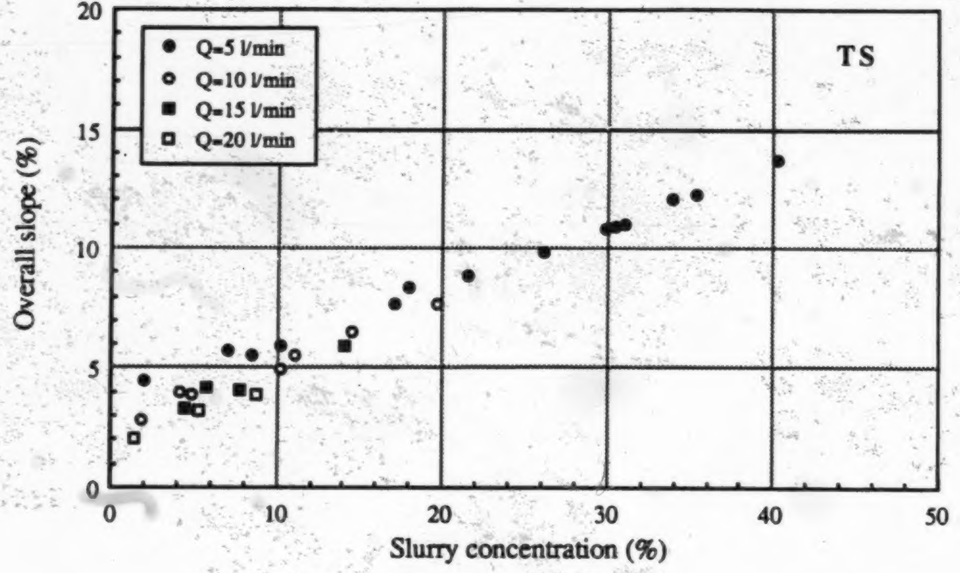


FIG. 3.27- Variation of the overall slope with slurry concentration for TS flume tests

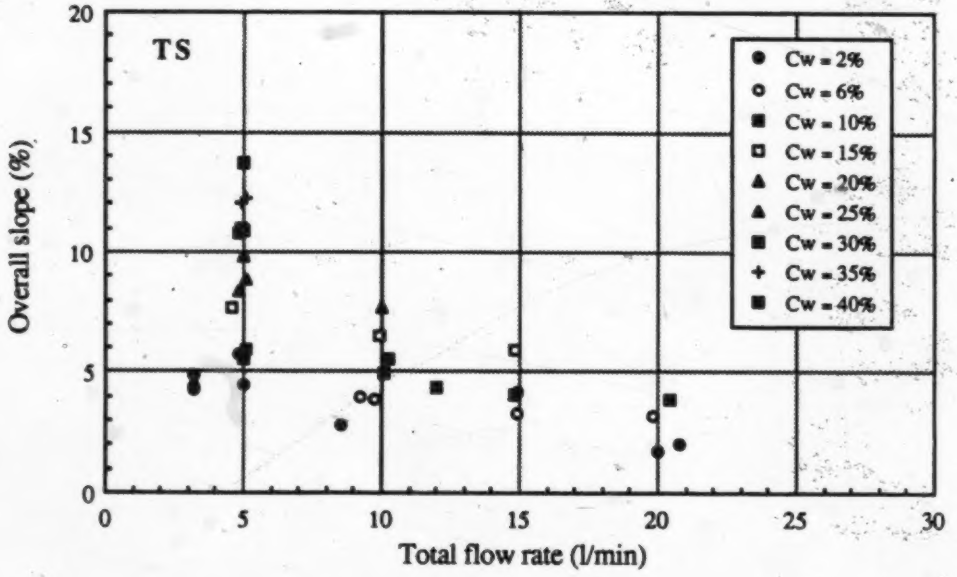


FIG. 3.28 - Variation of overall slope with total flow rate for TS flume tests

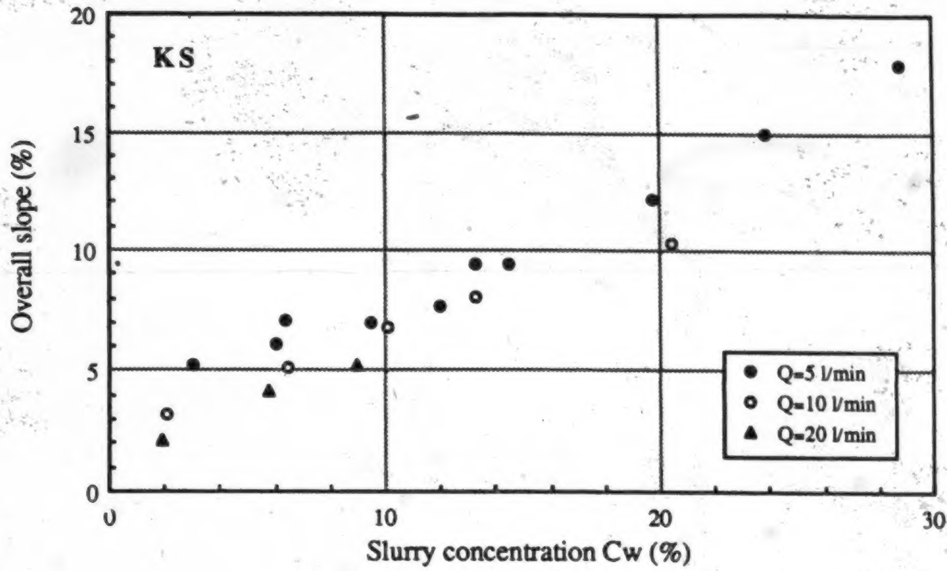


FIG. 3.29 - Variation of overall slope with slurry concentration for KS flume tests

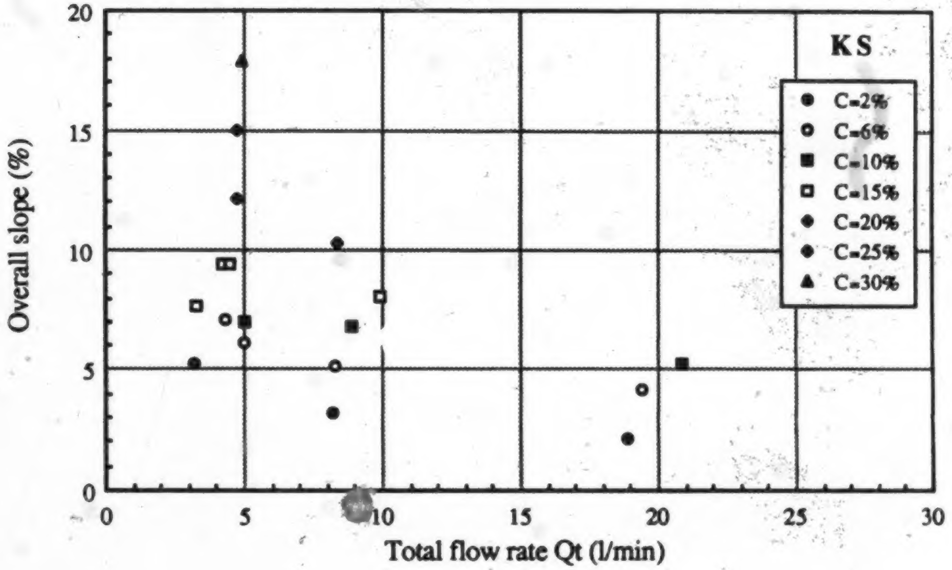


FIG. 3.30 - Variation of overall slope with total flow rate for KS flume tests

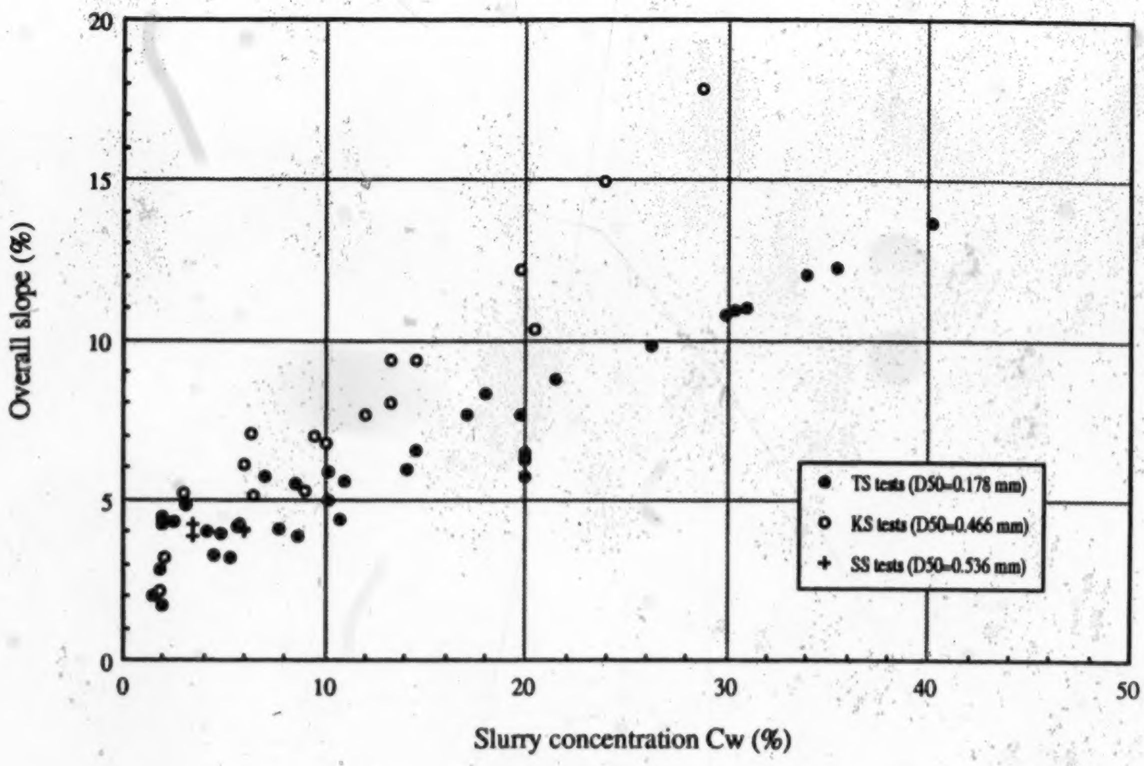


FIG. 3.31 - Variation of overall slope with slurry concentration for TS, KS and SS flume tests

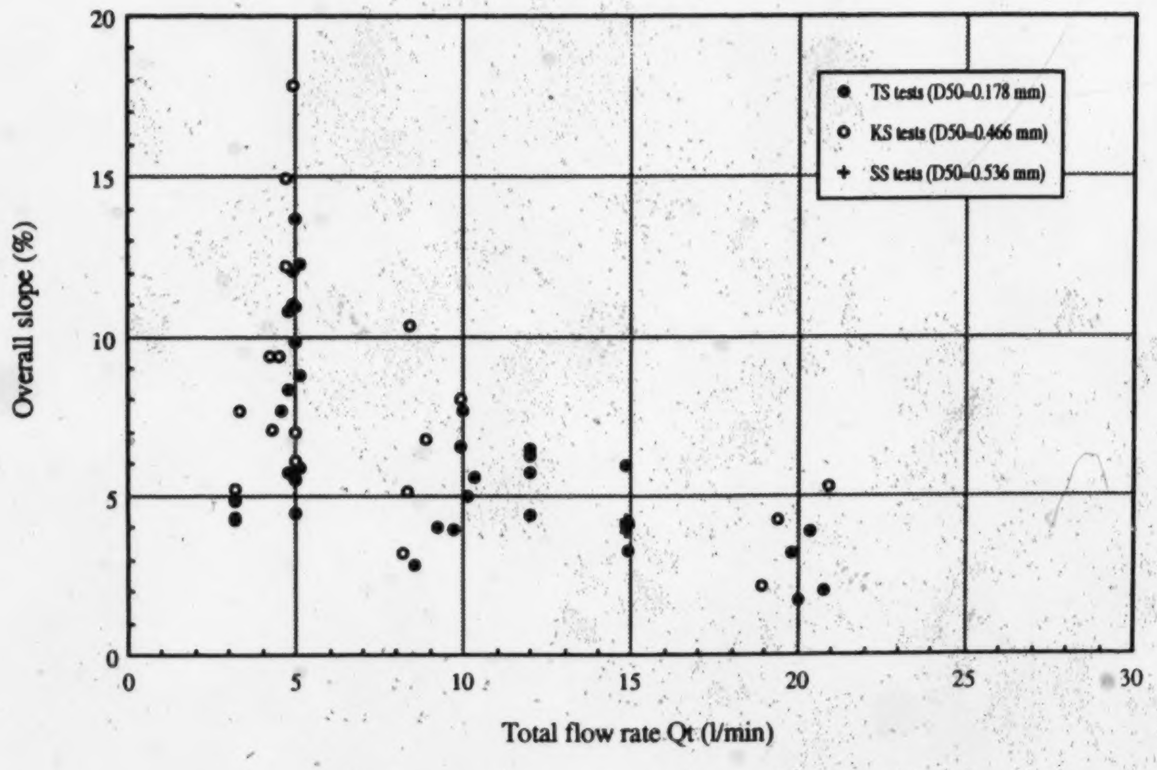


FIG. 3.32 - Variation of overall slope with flow rate for TS, KS and SS flume tests

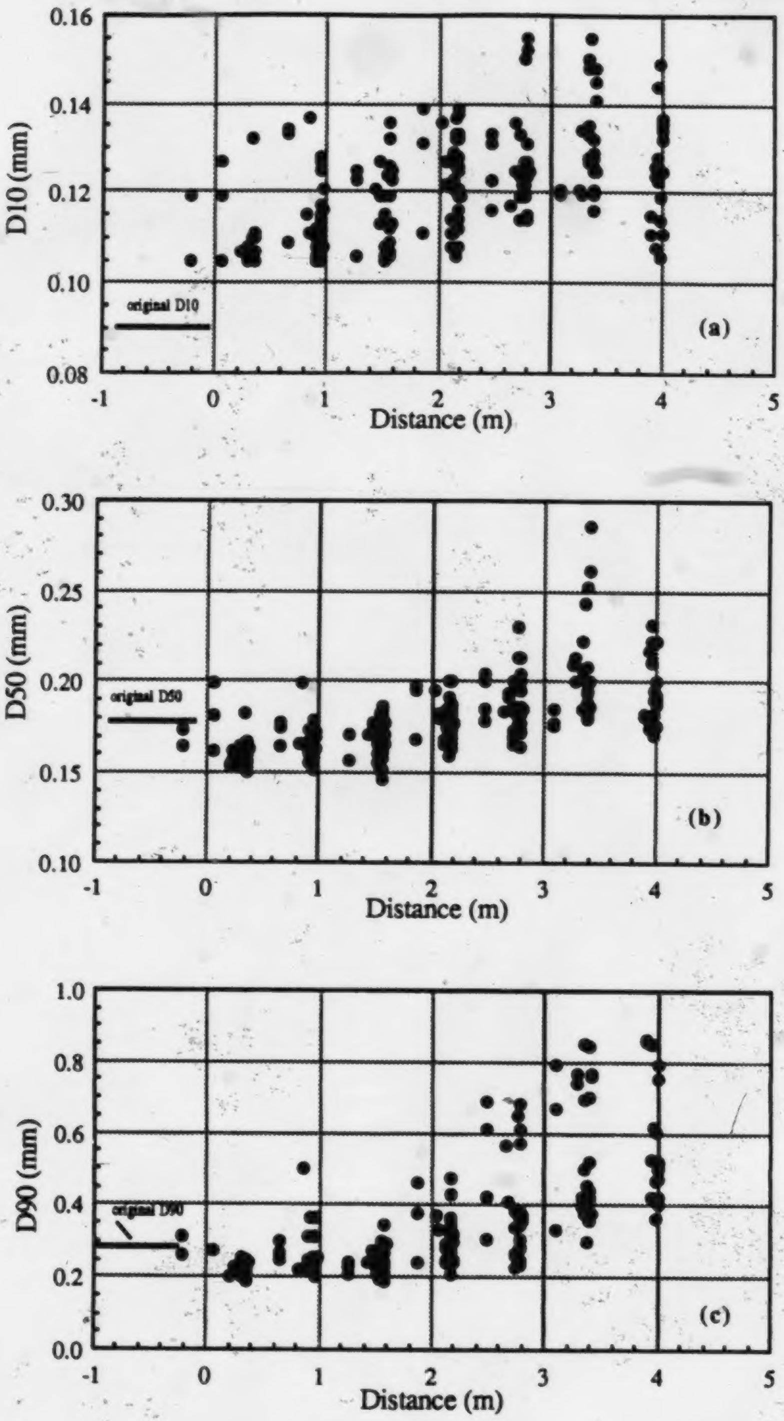


FIG. 3.33 - Variation of grain size parameters with distance for TS flume tests (note different scales)

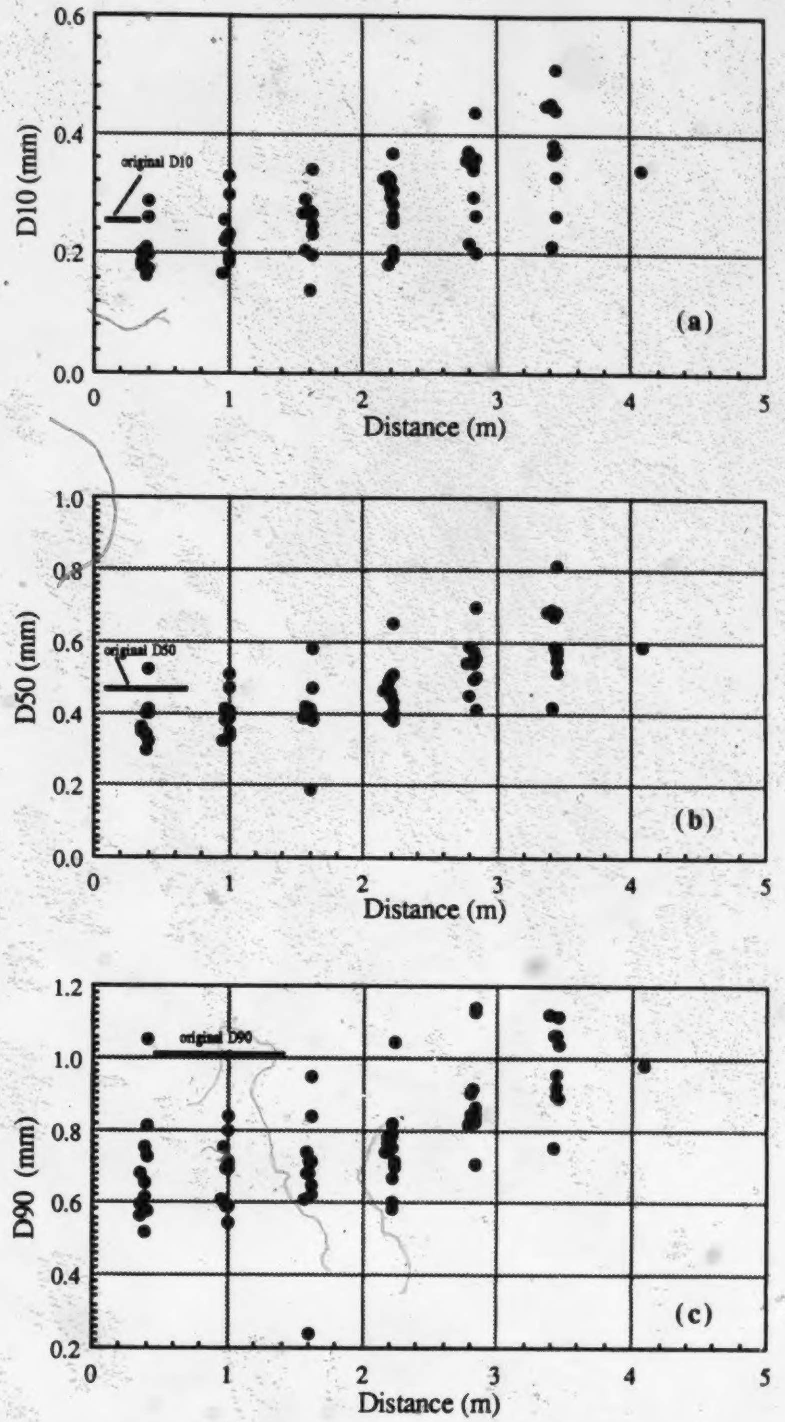


FIG. 3.34 - Variation of grain size parameters with distance for KS flume tests (note different scales)

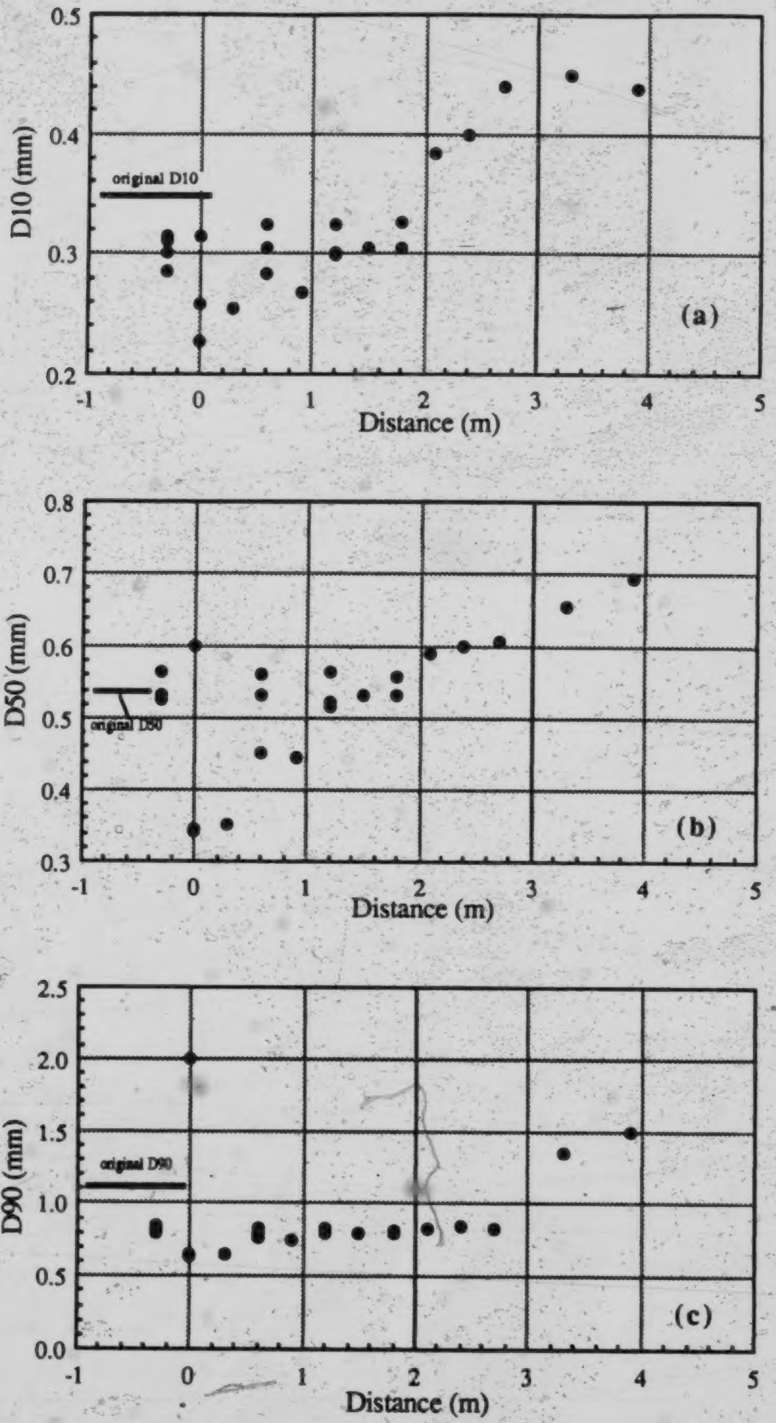
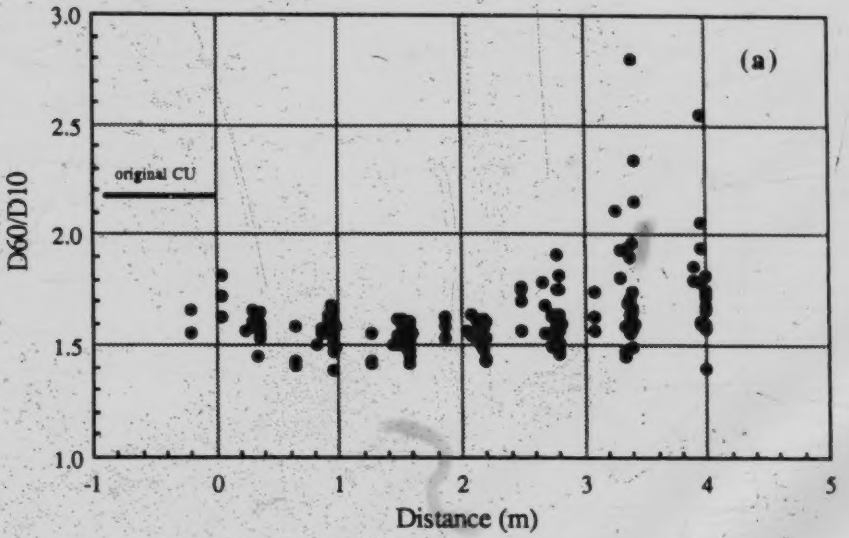


FIG. 3.35 - Variation of grain size parameters with distance for SS flume tests (note different scales)



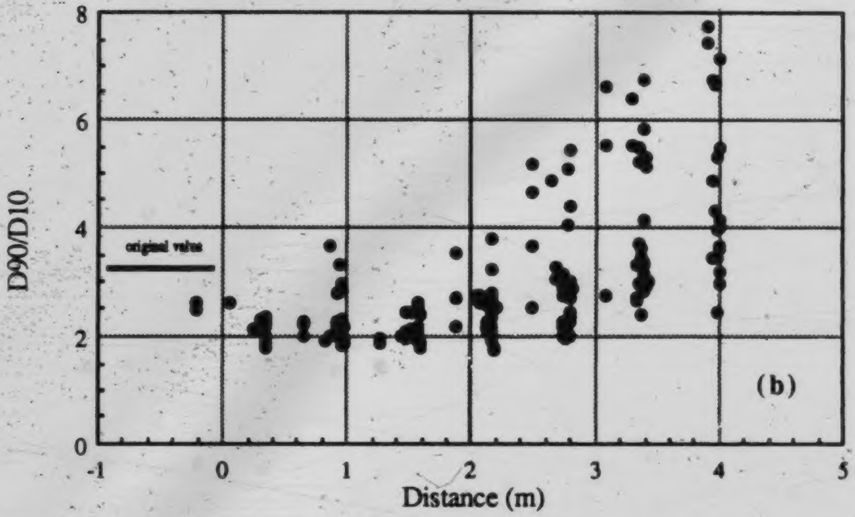
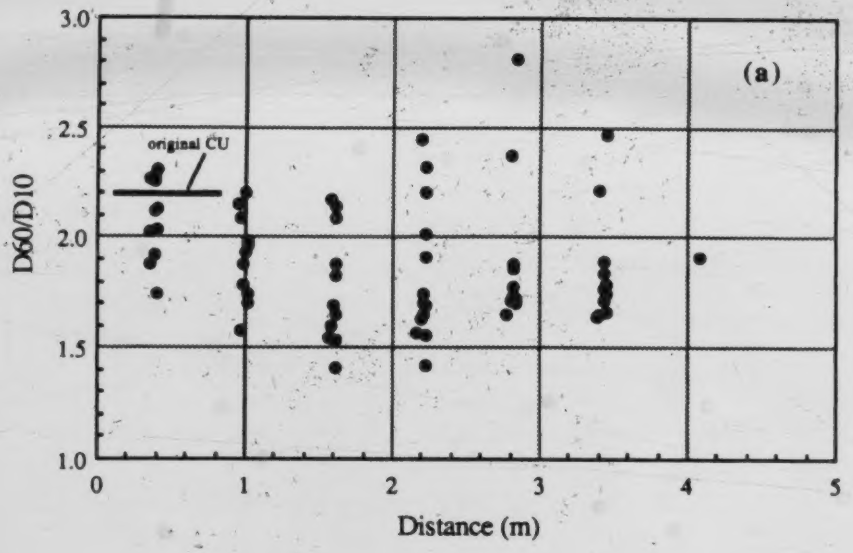


FIG. 3.36 - Variation of grain size parameters with distance for TS flume tests (note different scales)



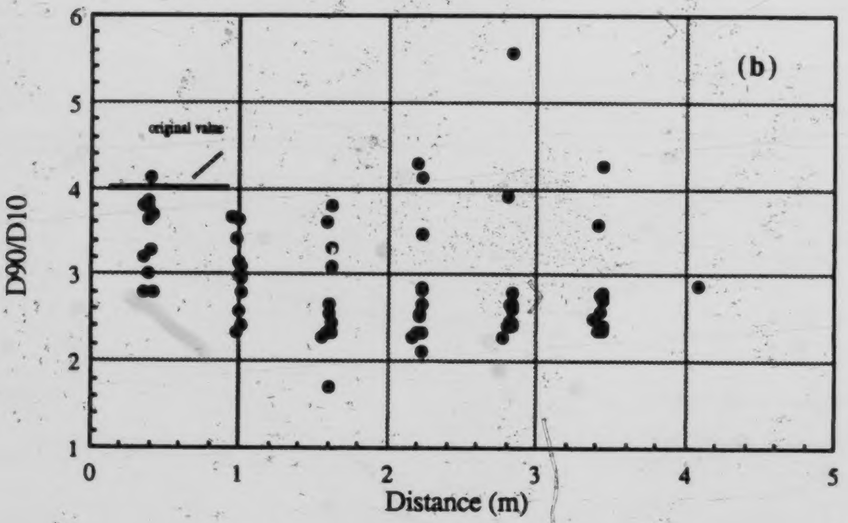
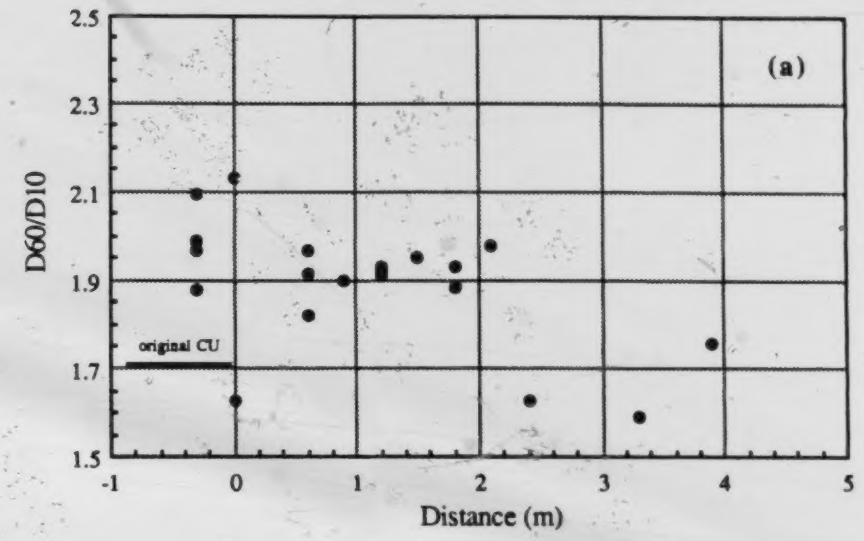


FIG. 3.37 - Variation of grain size parameters with distance for KS flume tests (note different scales)



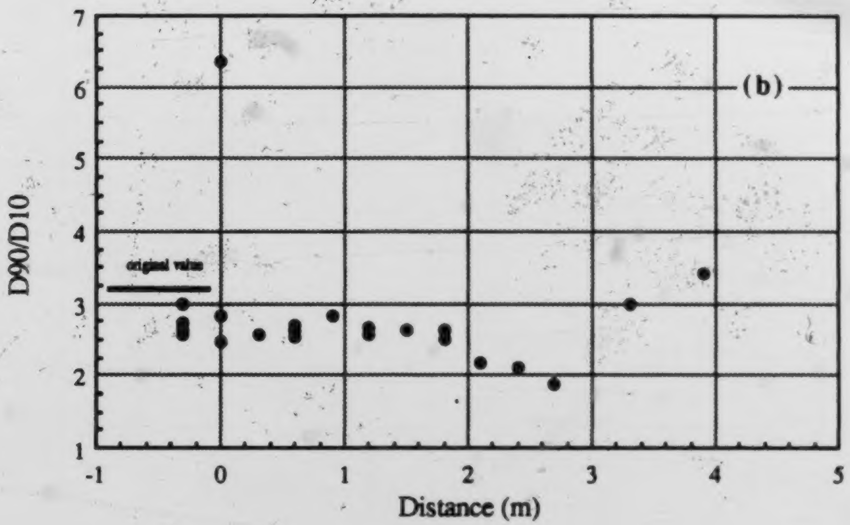


FIG. 3.38 - Variation of grain size parameters with distance for SS flume tests
(note different scales)

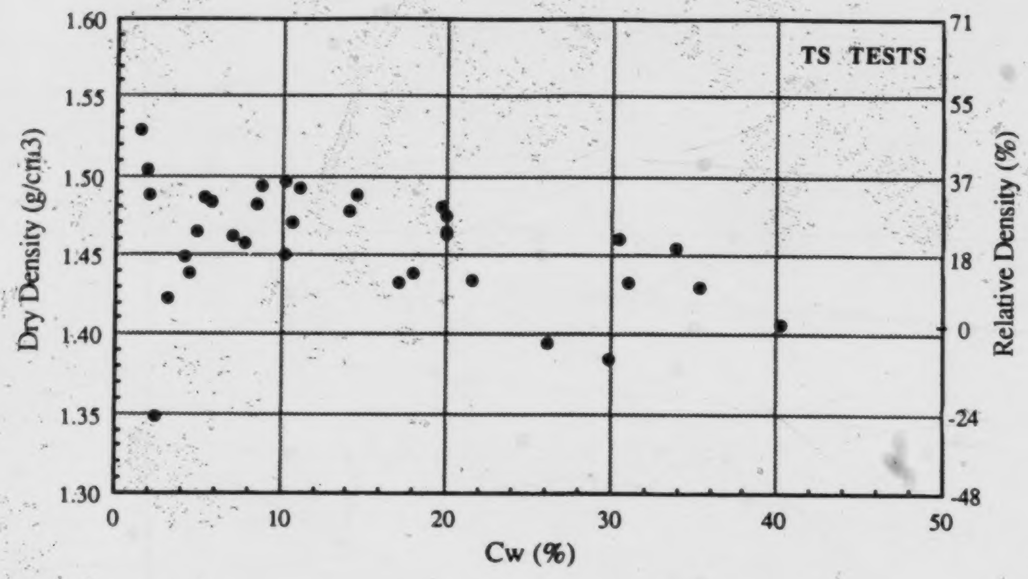


FIG. 3.39 - Variation of density with slurry concentration for flume tests using TS sand

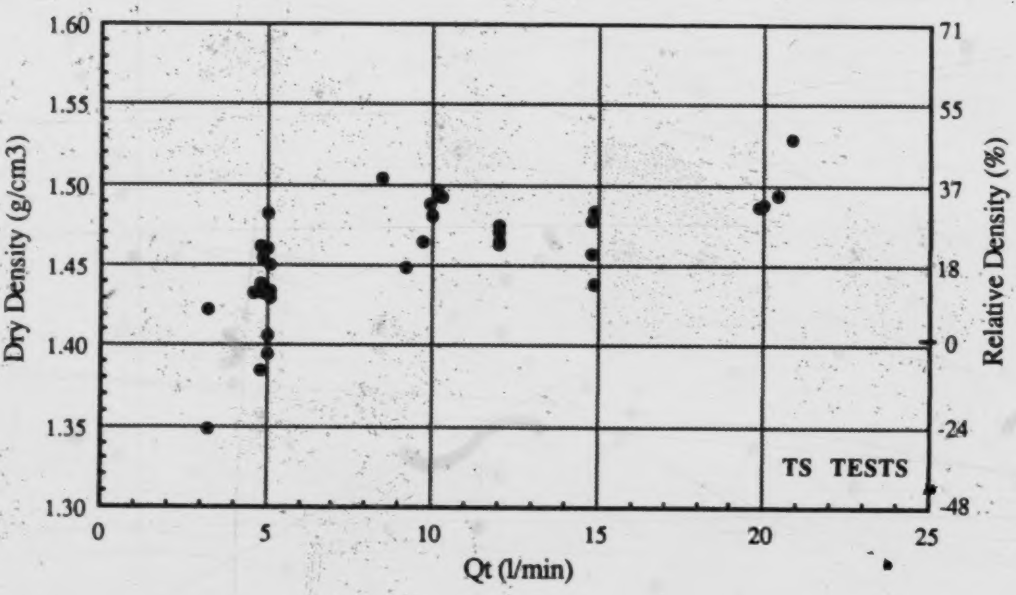


FIG. 3.40 - Variation of density with flow rate for flume tests using TS sand

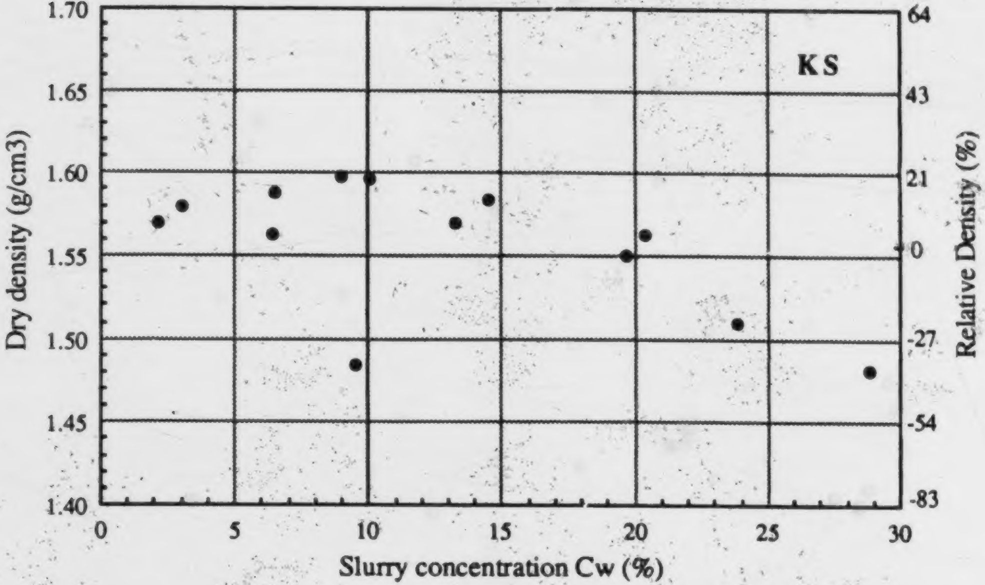


FIG. 3.41 - Variation of density with slurry concentration for flume tests using KS sand

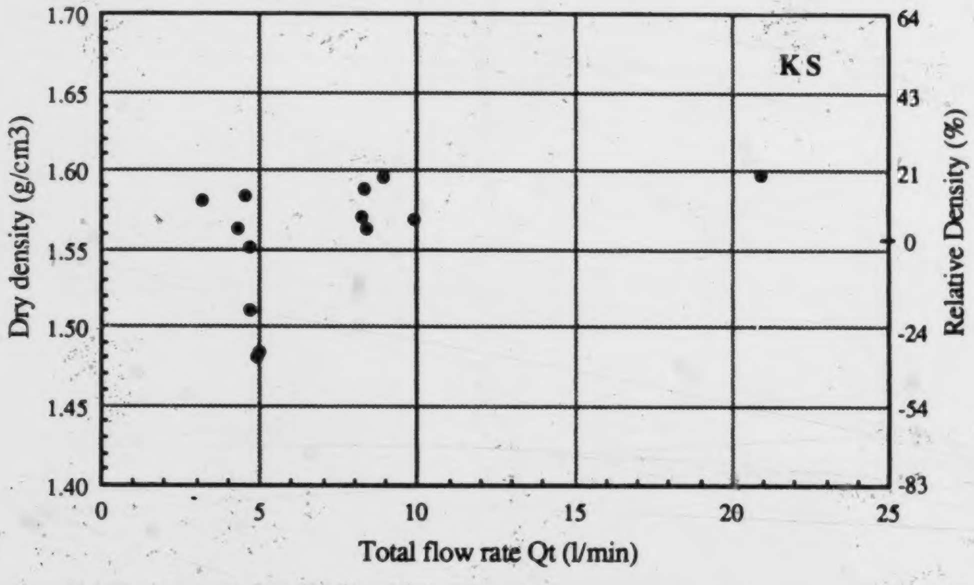


FIG. 3.42 - Variation of density with flow rate for flume tests using KS sand

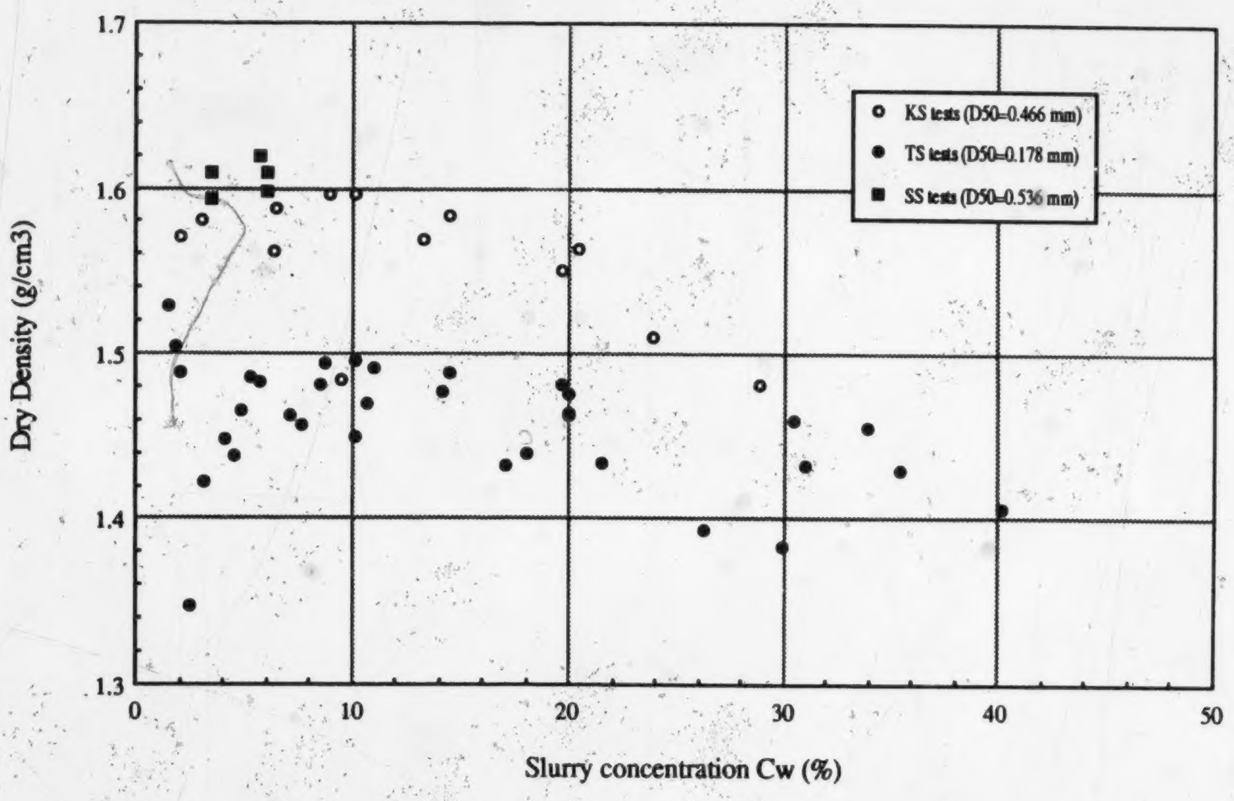


FIG. 3.43 - Variation of density with slurry concentration for TS, KS and SS flume tests

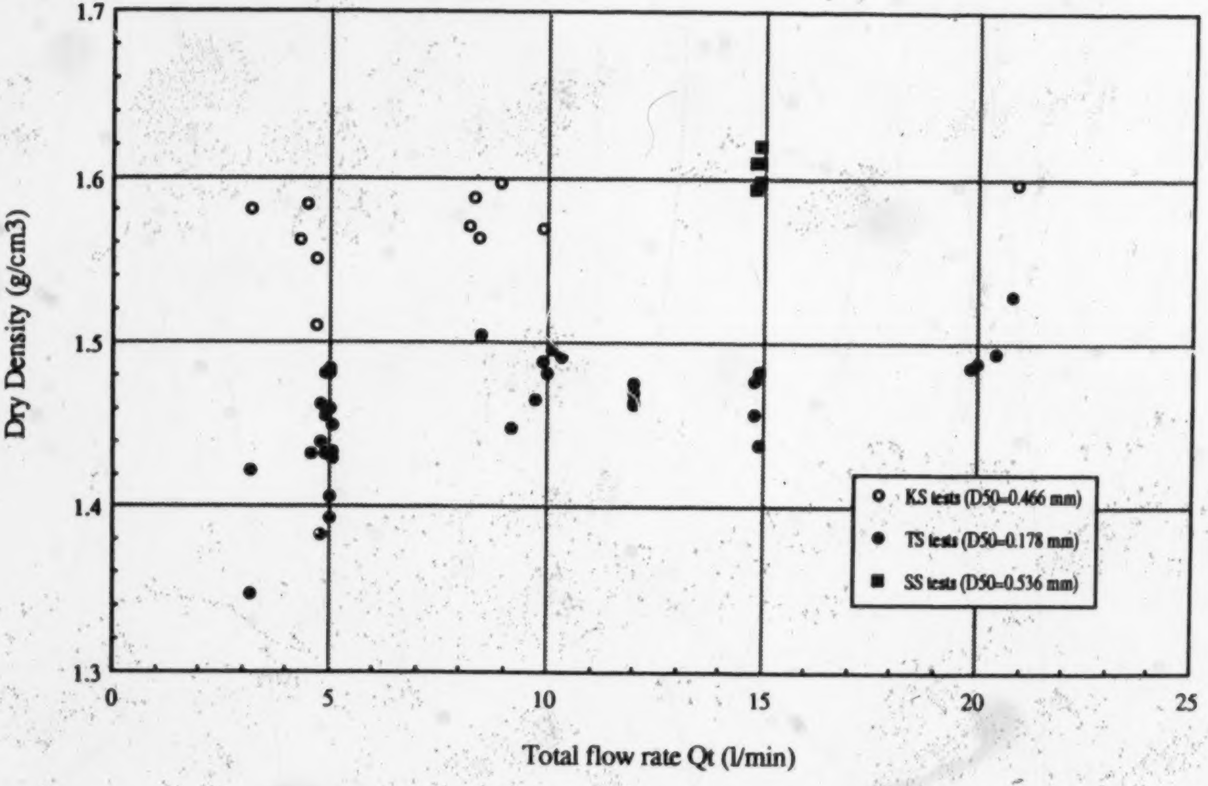


FIG. 3.44 - Variation of density with total flow rate for both TS, KS and SS tests

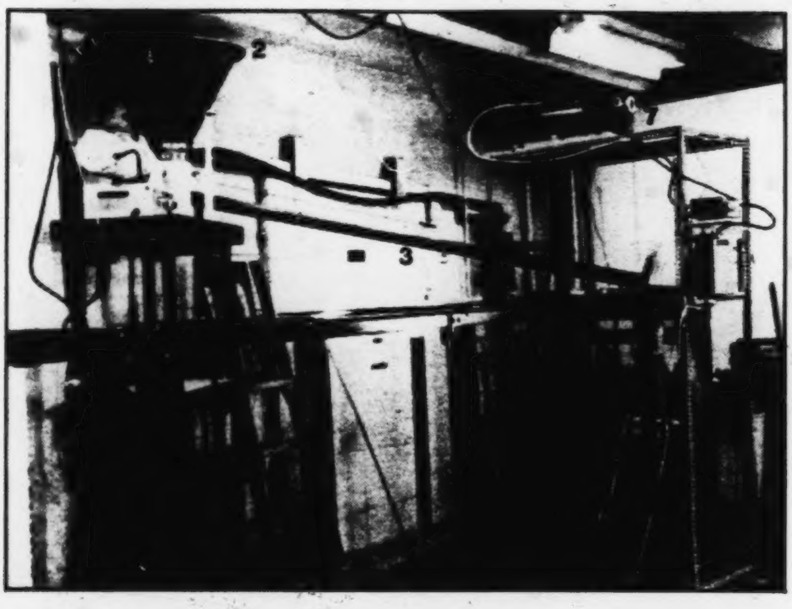


PHOTO 3.1 - Laboratory set-up (*2* sand feeder; *3* chute; *4* flow spreader; *5* funnel; *6* flexible hose; *7* electric motor; *8* floater)

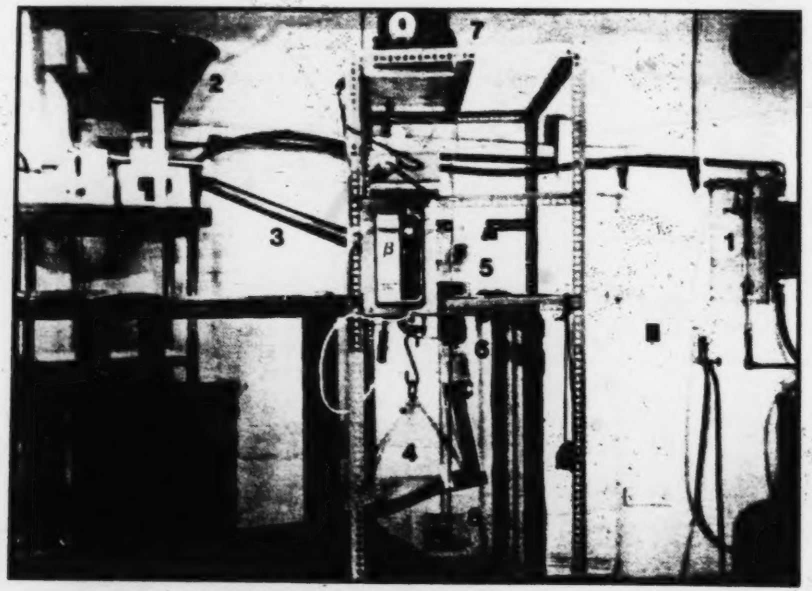


PHOTO 3.2 - Another view of the laboratory set-up (*1* flowmeter; *2* sand feeder; *3* chute; *4* flow spreader; *5* funnel; *6* flexible hose; *7* electric motor; *8* floater)

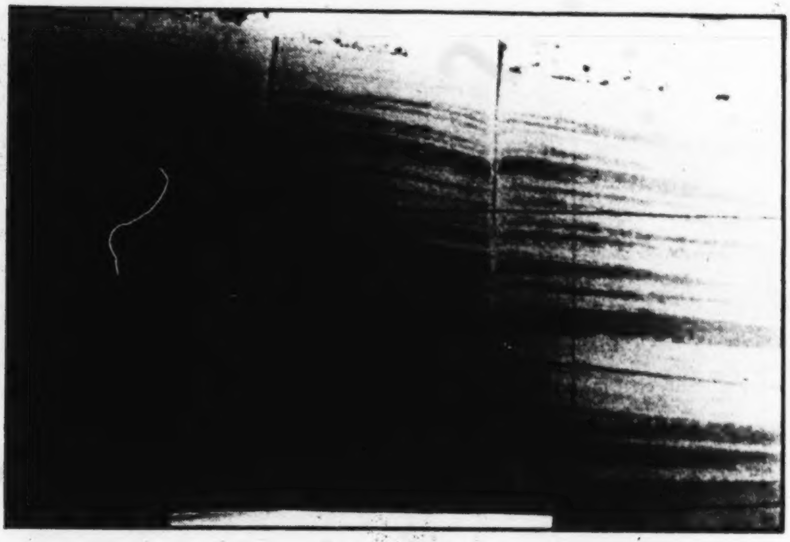


PHOTO 3.3 - Half sampler being pushed down against the flume wall to study sample disturbance

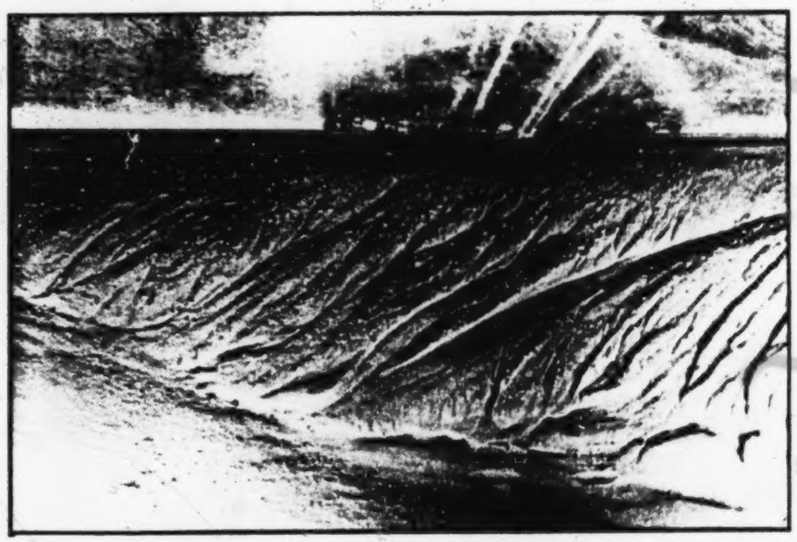


PHOTO 3.4 - Plan view of flume deposit showing current lineation and riffles (where shallow flow over bar meets channel)

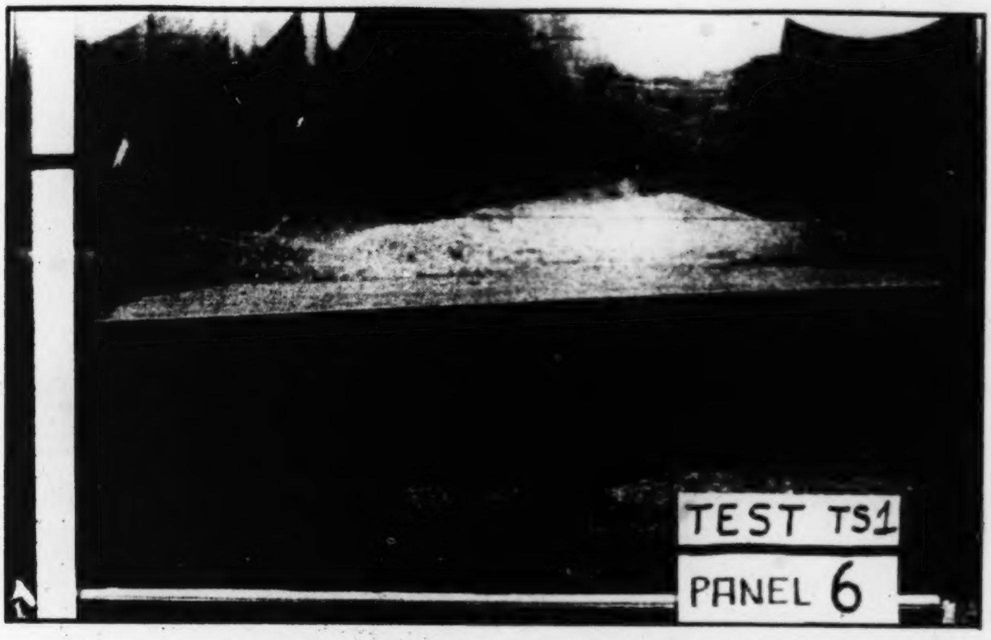


PHOTO 3.5 - Typical stratigraphy of the flume deposit showing lamination dipping upstream and cross-stratification, formed when the flow enters the pond

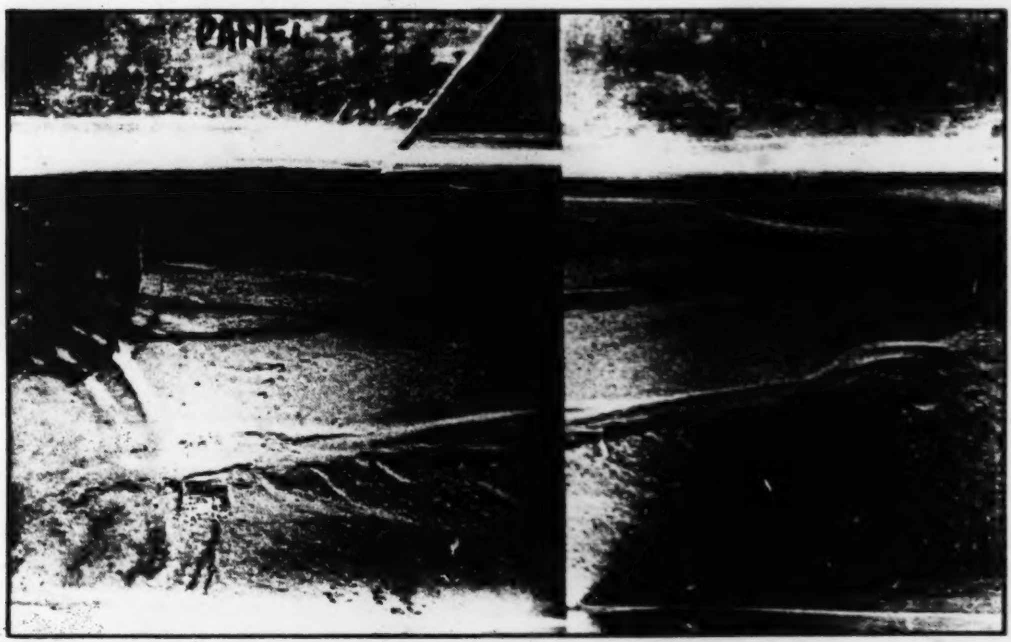


PHOTO 3.6 - Plan view of the downstream end of the flume deposit, showing slump of the underwater slope; note difference in grain size on bar and in channel

Chapter 4

Comparison between various flume tests used for hydraulic fill studies

4.1 - INTRODUCTION

Flume tests are a convenient tool to study hydraulic fills. Under laboratory conditions it is possible to control and isolate variables in a simpler and more economical manner than would be possible in the field. Flume tests also permit one to study the hydraulic deposition process of a certain material at an early stage of the project when information is necessary but field data are still not available.

Due to these advantages, several flume tests programs have been carried out to study hydraulic deposition. In fact, there is a large number of flume tests presented in the literature involving flow of water and sediment. However, the majority of these tests study hydraulic deposition from a hydraulics or sedimentology point of view. In these cases, a bed of the selected sediment is usually placed previously on the flume bottom and water flows over it at specified flow rates and velocities. The water flow may cause the sediment to be transported defining particular bedforms and stratification. The sediment that is carried out at the downstream end of the flume is usually collected and replaced at the upstream end, so the bed may change shape but it is not aggrading or degrading. In only a few of these flume tests (Allen, 1963; Bhamidipathy and Shen, 1971; Soni et al., 1977; Soni, 1981; Garde et al., 1981; Torres and Jain, 1984; Yen et al., 1989) is there net deposition with additional sediment being placed at the flume head. Typical measurements during these tests include water flow rate, velocities, depth, sediment transport rate and slope. Bedform and stratigraphy are sometimes also recorded.

Relatively few flume tests found in the literature deal with hydraulic fill from a geotechnical perspective. The objectives of these tests are, in general, the determination of the beach slopes and/or the study of physico-mechanical characteristics of the deposited material. Flume tests for hydraulic fill studies obviously have to involve an excess sediment being deposited and therefore an aggrading bed. The main difference in testing procedure between these tests and hydraulics/sedimentology aggrading bed tests is that in the first case water and solids are previously mixed and fed to the flume as a slurry, while in the latter case excess sand is dropped on to an already established flow (that is usually much deeper than the flow in slurry deposition tests).

In this chapter, flume tests performed specifically to study hydraulic fill are described and their results are summarized and compared.

4.2 - SUMMARY OF FLUME TESTS PRESENTED IN THE LITERATURE

A brief description of several flume tests to study hydraulic fills is presented in this section. A summary of the most important features and parameters utilized in each test program is presented in Table 4.1.

4.2.1 - Porto Primavera Dam, Brazil (UPP)

The flume tests reported by Ferreira et al.(1980) were conducted according to Soviet technology as part of the preliminary studies for the design and construction of Porto Primavera Dam in Brazil, which was planned initially to be a hydraulic fill. The Soviets report to have used flume tests extensively in the past when the hydraulic fill technology was being developed in their country. Nowadays the hydromechanization process is standardized in the USSR and laboratory deposition tests are no longer used routinely (Filimonov, 1979).

For the Porto Primavera Dam studies, fine sand from one of the borrow areas was deposited in a 11 m long flume using an independent sand and water feeding systems (Table 4.1). Sand was added at a controlled flow rate to a water flow just before being fed to the flume. Turbulence ensured the formation of a uniform slurry. The flume was provided with piezometers installed along its bottom and sides. The bottom of the flume had a drainage system with valves that allowed drainage of the fill at a rate of 4 cm/h after the deposition stopped. The slurry was discharged from a vertical pipe with a series of outlets installed at different heights. Each outlet was a short piece of pipe pointing upwards (possibly to minimize the flow velocity) and plugged. As the fill rose, the outlets were successively opened.

Three tests were carried out with slurry concentration around 10% by weight and specific flow rates (flow rate divided by flume width) varying from 3.3 to 13 cm^3/s cm. The formation of meanders is reported in some cases. Measurements during the tests included concentration of the slurry being discharged and concentration and composition of the outflow of the "pond's spillway". After the tests, the final sand profile was recorded and undisturbed samples were taken for determination of density and grain size distribution of the sand along the flume at two depths. Horizontal and vertical undisturbed samples were used to determine permeability, compressibility and shear strength on directions parallel and perpendicular to the stratification. The sand deposited in the flume had a relative density in the range of 50 to 65% and some anisotropy of the permeability with K_h/K_v varying between 1 and 10. Remolded samples showed lower permeability and less compressibility than the undisturbed flume samples. Shear strength was similar for samples from all three tests.

4.2.2 - Lakefield (LK)

Lakefield Research performed flume tests in 1983 to obtain slope data for the design of East Kemptville tailings dams in Nova Scotia, Canada (Lighthall, 1987). A series of tests was conducted with varying flume slopes and slurry concentration. However, the only data available are the grain size distribution curve of the coarse tailings used for the tests and the profiles of two tests using slurry concentrations of 20 and 45%. The flow rate is not known and there is no information about the test procedure, except that there was no water ponding at the slope toe. The slopes obtained were reported to be much steeper than the field slopes of the same material.

4.2.3 - South Africa (B)

The flume tests conducted by Blight et al.(1985) in South Africa had the objective of studying tailings beach profiles based on the concept of master profile proposed by Melent'ev et al.(1973) in which a normalized profile exists independent of the test scale. Three different gradations of silty tailings were deposited at three different slurry concentrations each (50, 60 and 70 % by weight) in a small flume (Table 4.1). The feeding system consisted of a 220 litre drum with a bottom discharge, but no details are provided on how the slurry was kept homogeneous in the drum during the test, nor on the testing procedure or flow rates. The only data reported is the final normalized profile for each test.

4.2.4 - U.S. Bureau of Mines (USA and USB)

Flume tests were also performed at the US Bureau of Mines Research Center in Spokane, Washington, USA (Boldt, 1988). Tailings obtained from two mine sites were deposited in a wooden sloping flume (Table 4.1). Tailings A consisted of a fine mill waste from a copper-silver mine with an average diameter of 0.0135 mm. Tailings B was a slightly coarser tailings (D_{50}= 0.097 mm) from a silver-lead-zinc mine. The bulk tailings were diluted with water in a 6400 litre mixing tank to form the slurry. The slurry was then pumped into the flume at controlled flow rates and concentrations, varying from 58 to 130 l/min and from 20 to 57% respectively. Boldt (1988) reports accentuated wall effects with the formation of side eddies that disturbed the flow in the flume. After the deposition was completed, the deposit was partially drained and Shelby tube samples were taken at designated distances along the length of the deposited tailings. The samples were used to determine shear strength, permeability and grain size distribution. The average beach slope was also measured for each test.

4.2.5 - Delft, The Netherlands (DS, DL1 and DL2)

Flume tests were carried out in Delft, The Netherlands, as part of experimental and theoretical studies related to the Deltaworks in the Southwestern part of the country, where various sea arms were closed using hydraulic fill techniques (de Groot et al.,1988; Winterwerp et al.,1990).

Two flumes of different sizes were used to deposit three gradations of fine to medium sand (Table 4.1). Slurry concentrations between 32 and 68% and flow rates varying from 7 to 35 l/min were utilized for the tests on the small flume (DS). The large scale flume tests (DL1 and DL2) covered a wide range of slurry concentrations (0-64%)

and used very large flow rates (180 to 2700 l/min), which is out of the range of the values used on other flume tests presented in the literature.

The equilibrium slope was defined in these studies as the slope at which sedimentation and erosion are in balance. It was determined by decreasing the slope of the tilting flume in small steps of about 0.001 rad until sand bars started growing on the bottom of the flume. The previous slope was then defined as the equilibrium slope. Therefore these tests differ from the others presented in the literature by the fact that no material was deposited in the flume. It is of relevance to note that the flumes had sand grains glued to the bottom, however for the large scale tests (DL1 and DL2) the sand glued on the bottom was much coarser than the sand being tested.

Sand concentration, flow rate, slope, flow depth and slurry temperature were measured on the small scale flume tests. In the larger scale tests the following parameters were determined: flow rate, temperature, concentration, slope, flow depth, flow velocity and sand concentration profiles at various locations. These tests did not deal with the geotechnical characteristics of hydraulically deposited materials, but they involved sophisticated hydraulic measurements. More details on these tests can also be found in Mastbergen et al.(1988) and Bezuijen and Mastbergen (1988).

4.2.6 - University of Queensland, Australia (FB, FN, FFC and FCC)

The flume tests reported by Fourie (1988) were carried out using a slurry tank adapted with an electrical agitator to feed a small flume. The discharge device consisted of a horizontal pipe to spread the flow across the flume and an energy dissipator to minimize the formation of a plunge pool at the discharge point. A very small flow rate was adopted for all tests (Table 4.1). Three types of tailings were tested: bauxite from North Queensland (FB), nickel ore slurry from New Caledonia (FN) and two gradations of coal tailings from South-East Queensland, a fine (FFC) and a coarse (FCC). These tests will not be compared with the others as most of them deposited a non-segregating slurry, thus having a distinct rheology. In non-segregating slurries the solids and the carrier fluid do not behave independently, but act as a viscous fluid. Therefore the physical phenomena involved are distinct from the cases of segregating slurries, which were used in the other experimental programs.

4.2.7 - University of Alberta, Dept. of Chemical Engineering (F)

The objectives of the flume tests carried out by Fan (1989) were to study the variation of beach profile with time and distance and the effects of slurry concentration and discharge on these profiles.

These tests were performed using an experimental procedure common to flume tests for hydraulic or sedimentology studies, where sand is fed independently to a flume where a flow of water is already established. However, a very shallow flow depth was adopted, as occurs in hydraulic fills. These tests covered a relatively narrow range of low concentrations and low flow rates. The data produced consisted of sand profiles at four different times during the tests, which had a duration of 20 min each. The flume dimensions and the sand and flow characteristics utilized are presented on Table 4.1.

4.2.7 - University of Alberta, Dept. of Civil Engineering (SS, TS and KS)

Three different sands were tested in this work to study the influence of flow rate, slurry concentration and grain size distribution on the properties of hydraulic fills. These tests used a feeding system where water and sand were fed independently in a chute and formed the slurry on the way to the flume. The discharge device consisted of a flow spreader capable of distributing the flow uniformly across the flume. An automatic system was designed to keep the flow spreader at a constant height from the rising fill.

The slurry concentration was varied between 1.5 and 40.5% by weight and the flow rate from 50 to 350 cm^3/s (3 to 20 l/min) (see Table 4.1).

4.3 COMPARISON OF RESULTS

The various flume tests described in the previous section had the general objective of studying hydraulic fill, however the specific objectives were slightly different in each case. As a result, the variables that were studied and the parameters that were measured were not necessarily the same for all test programs. Moreover, there were differences in experimental procedure, some of which may have influenced the results obtained. In particular, the experiments carried out by the Delft group (de Groot et al.,1988; Winterwerp et al.,1990) and by Fan (1989) must be singled out for using very distinct techniques:

a) an equilibrium slope defined as the slope at which no deposition or erosion occurs was adopted for the tests performed in Delft since this slope is considered to be similar to the field slopes (Winterwerp et al.,1990). Consequently no material is deposited in the flume and no data is available on fill properties. Also, the slurry flows on the flume bottom flat surface instead of on the concave surface typical of other tests, and for the large scale tests (DL1 and DL2), it flows on a bed of grains of mean diameter 2 to 4 times larger than the mean grain diameter of the grains in the slurry, while for other flume and field cases the slurry flows over grains of size similar to its solid fraction.

b) Fan (1989) adopted a procedure of feeding sand to an established flow of water instead of discharging a slurry. However, the use of very shallow flow depths (less than 1 cm), relatively low slurry concentrations and no fines in the sand may contribute to minimize any potential influence of the different procedure on the results obtained.

Slurry concentrations adopted for the various tests varied between 0 and 70% by weight (Table 4.1). This is a wide range that covers the typical values of slurry concentration utilized for tailings dams (30 to 55%), for fill construction in the Soviet Union (10 to 20%) and for dredging operations in the Netherlands (25 to 60%). The range of values of flow rate and specific flow rate (flow rate divided by flow width) is also presented in Table 4.1. Note that the flow rates used for the large scale flume tests performed in Delft (DL1 and DL2) were much larger than all other values.

The following items compare specific aspects of the various flume tests such as type of solid material being deposited, geometry of the resulting fill and grain size distribution and density of the deposit.

4.3.1 - Materials

The grain size distribution curves of the materials discharged in the various flume tests described in the previous section are presented in Figure 4.1. Most of these soils can be classified as fine to medium sands, with mean grain diameters varying between 0.084 and 0.536 mm. Exceptions are USBM tailings A (USA) and Blight's fine and total tailings that are medium to coarse silts. Compared to the criteria of suitability for hydraulic fills proposed in the Soviet standard specification for construction of hydraulic fill dams SNiP-II-53-73, most of these sands belong to the Group I (Figure 4.2) which corresponds to recommended materials for construction of homogeneous hydraulic fills, an "ideal" material. Group II includes the materials that are recommended for hydraulic fill construction if the coefficient of uniformity (C_U) is high. Groups III and IV comprise material that have restricted application due to the high amount of fines. According to the Soviet code, materials in Group III (such as Blight's materials and USBM tailings B) require slow construction rate to avoid instability and Group IV materials (such as USBM tailings A) cannot be used with a structural function. The coarse materials in Group V are also of restricted applicability due to excessive permeability but they can be used for separate placement of dam shoulders.

Grain size analyses of the material already deposited in the flume are presented by Ferreira et al.(1980), Boldt (1988) and in this work (Chapter 3). The results from Ferreira et al.(UPP) and Küpper (SS, TS and KS) show limited hydraulic sorting with mean grain size increasing with distance from the discharge point. Using much less uniform materials, Boldt (USA and USB) found a decrease in mean grain size along the flow.

When comparing tests carried out using different materials, it is important to note that grain shape, angularity and surface features may affect the results and may be responsible for some apparent scatter of the data.

4.3.2 - Geometry

All flume tests that utilized segregating slurries, and for which profiles were reported, formed fills with a similar concave shape as shown in Figures 4.3 to 4.5. The influence of the input parameters (such as particle size, slurry concentration and flow rate) on the fill geometry is evident in these figures.

In many cases only overall slopes were presented rather than actual profiles. The overall slope is (or is assumed to be) defined as the maximum height (at the discharge point) divided by the distance from the point of maximum height to the end of the beach and expressed as a percentage. Slurry concentration is defined as the weight of solids in the slurry divided by the total weight.

The effect of the mean grain size and the slurry concentration on the average slope of the fill is shown in Figure 4.6. This graph compares results of tests described in Chapter 3 on TS tailings sand with the results from Fan (1989) on a commercial sand. All of these tests were carried out using the same flow rate ($Q = 250$ cm^3/s) and the same specific flow rate ($q = 8$ cm^3/s cm) and show that the average slope increases with the mean grain size and with slurry concentration.

Figure 4.7 presents the variation of average slope with concentration of the slurry being deposited in various flume tests. The relatively large scatter in this graph is partially due to the inclusion of points corresponding to a wide range of grain sizes and flow rates. The flat slopes of the large scale flume tests performed in Delft (DL1 and DL2) can be explained as caused by the much higher flow rates utilized in this case compared to the remainder flume tests. And the flat slopes obtained in the tests carried out by Boldt (1988) (USA and USB) can be attributed to the use of materials containing a high percentage of fines (90% and 45% finer than 74 μm, respectively). Limiting this analysis to sandy fills deposited at flow rates under 2500 cm^3/s and specific flow rates less than 40 cm^3/s cm in order to better isolate the effect of the slurry concentration, a much smaller scatter is observed (Figure 4.8) and the increase in average slope with concentration becomes more evident. The shaded zone in Figure 4.8 includes more than 90% of the experimental points.

An increase in flow rate or specific flow rate causes the slopes to become flatter and this tendency is shown in Figures 4.9 and 4.10. The small scale tests carried out in Delft (DS) yielded more scattered results featuring relatively steep slopes for the correspondent values of flow rate and, especially, specific flow rate. The results of the companion large scale flume tests are not presented as the much larger flow rates utilized in these tests would have obscured the other results.

4.3.3 - Density

The only values of density of flume deposited materials that were found in the literature were the results presented by Ferreira et al.(1980) (see Table 4.1). Figures 4.11 and 4.12 compare the results measured for UPP (Ferreira et al.,1980) with the results obtained for the SS, TS and KS series (Chapter 3). The scatter of the data is partially because the plot of density versus slurry concentration includes tests using a range of flow rates and similarly the points in Figure 4.12 correspond to different slurry concentrations. Also, other factors associated with the data scatter are difficulties in obtaining accurate density measurement of undisturbed samples of relatively clean sands, variations in grain size distribution curves and effects of grain shape, angularity and surface features.

Compared to SS, TS and KS values, the densities of UPP tests seem a little high for a material with a lower D_{50} .This fact could have being caused by several factors including:

a) a difference in mineralogy and/or grain shape among the materials;

b) the higher coefficient of uniformity (CU) of UPP material compared to TS and KS sands (voids can be better filled); and

c) the lower discharge velocities adopted for UPP tests in relation to the values used for SS, TS and KS tests.

Results from SS, TS, KS and UPP tests show the same trend of variation of density with flow rate and slurry concentration: density decreases as slurry concentration increases and density tends to increase as specific or total flow rates increase. This is consistent with the trends reported by Yufin (1965).

4.4 - SUMMARY AND CONCLUSIONS

Several flume tests carried out in different parts of the world to study hydraulic fills were compared in this chapter. These tests were performed using slightly different testing procedures and covered a wide range of values of slurry concentration and flow rate. Most of the tests deposited sand, but some of them involved very fine tailings materials.

The results of all test programs show a consistent trend of fill slopes becoming steeper as the flow rate decreases and as the slurry concentration and the mean grain size increase. Although more limited, the density data point to an increase in fill density as the flow rate increases and a decrease in density for higher values of slurry concentration.

Generally, these conclusions are consistent with observations of hydraulic fills and natural alluvial deposits which suggests that, at least qualitatively, flume tests are adequate to simulate the physical phenomena associated with hydraulic deposition in the field.

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TABLE 4.1 - Summary of flume tests for hydraulic fill study reported in the literature

REFERENCES	TEST CODE	# of tests	Flume length l (m)	Flume width w (m)	l/w	Flume depth d (m)	Feeding System	Flume wall material	D50 (mm)	CU	D90/D10	Qt(vol) (cm3/s)	q=Qt/w (cm2/s)	Veloc Vo(cm/s)	Cw(%) Ws/Wt	Comments
Ferreira et al.(1980)	UPP	3	11.0	0.8	14	>0.8	independent	1 side acrylic	0.140			260-1040	3.3-13.0	13 - 53	6.7-9.5	controlled bottom drainage (4 cm/h)
Lakefield (1983)	LK	> 2	>1.5	N/A	N/A	> 0.15	N/A	N/A	0.331	3	4.5	N/A	N/A	N/A	20 & 45	
Blight et al.(1985)	B	>9	1.8	0.3	6	0.6	220 l drum	Perspex	0.084 0.042 0.023	27 22 17	59 45 47	N/A	N/A	N/A	50 - 70	
Boldt (1988)	USA USB	12	12.2	0.6	20	0.6	mixing tank	plywood	0.014 0.097	8 22	30 55	965-2172 1207-2172	15.8-35.6 19.8-35.6	122-274 152-274	26 - 57 20 - 50	wall effects influenced results
de Groot et al.(1988) & Winterwerp et al.(1990)	DS DL1 DL2	>50 >20	1.5 9.0	0.118 0.3	13 30	0.045 0.3	mixing tank recirculatio	N/A N/A	0.134 0.120 0.225	N/A N/A N/A	N/A N/A N/A	118-590 3000-45000	10-50 100-1500	13-83 100-190	32 - 68 0 - 64	no deposition flow on flume bottom (DS:D50b=0.134mm) (DL:D50b=0.500mm)
Fourie (1988)	FB FN FFC FCC	2 2 4 1	2	0.6	3	0.6	mixing tank	Perspex	0.149 0.023 0.008 0.149	2.4 2.1 >30 9.1	9.6 8 >240 40.9	8.3	0.14	2.7	27&32 35&42 15 to 50 37	most presented non-segregating slurry behaviour
Fan (1989)	F	8	4.9	0.3	16	0.5	independent	Plexiglass	0.267	1.7	2.2	114-280	3.7-9.0	N/A	8 - 14	sand deposited into flowing water
KUPPER (1991)	SS TS KS	5 37 19	6.1	0.3	20	1.2	independent	Plexiglass	0.536 0.178 0.466	1.7 2.2 2.2	3.2 3.2 4.0	53-348	1.8-11.6	30-80	1.5-40.4	pervious foundation some bottom drainage

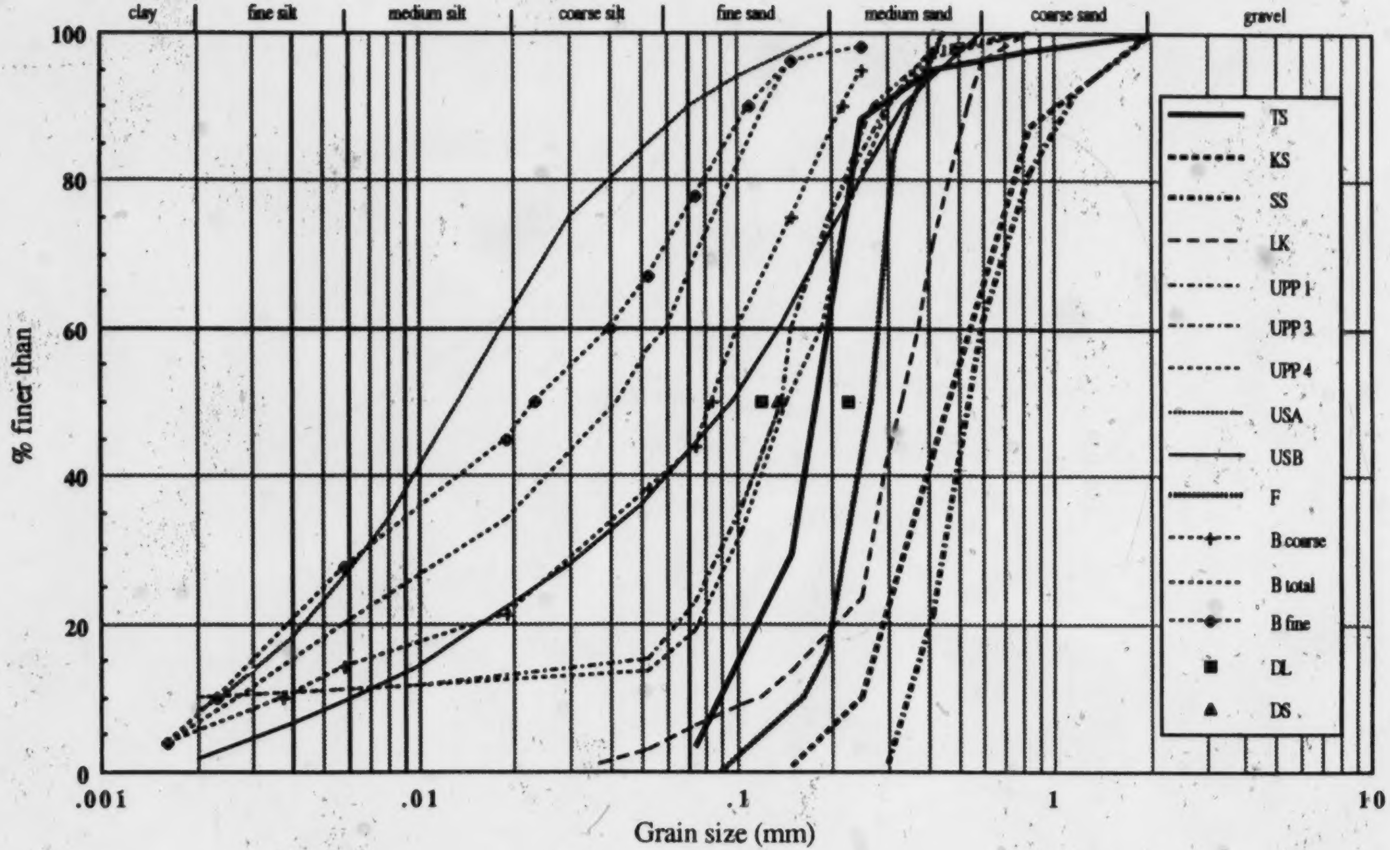

FIG. 4.1 - Grain size distribution curves of materials used in various flume test programs (see Table 4.1)

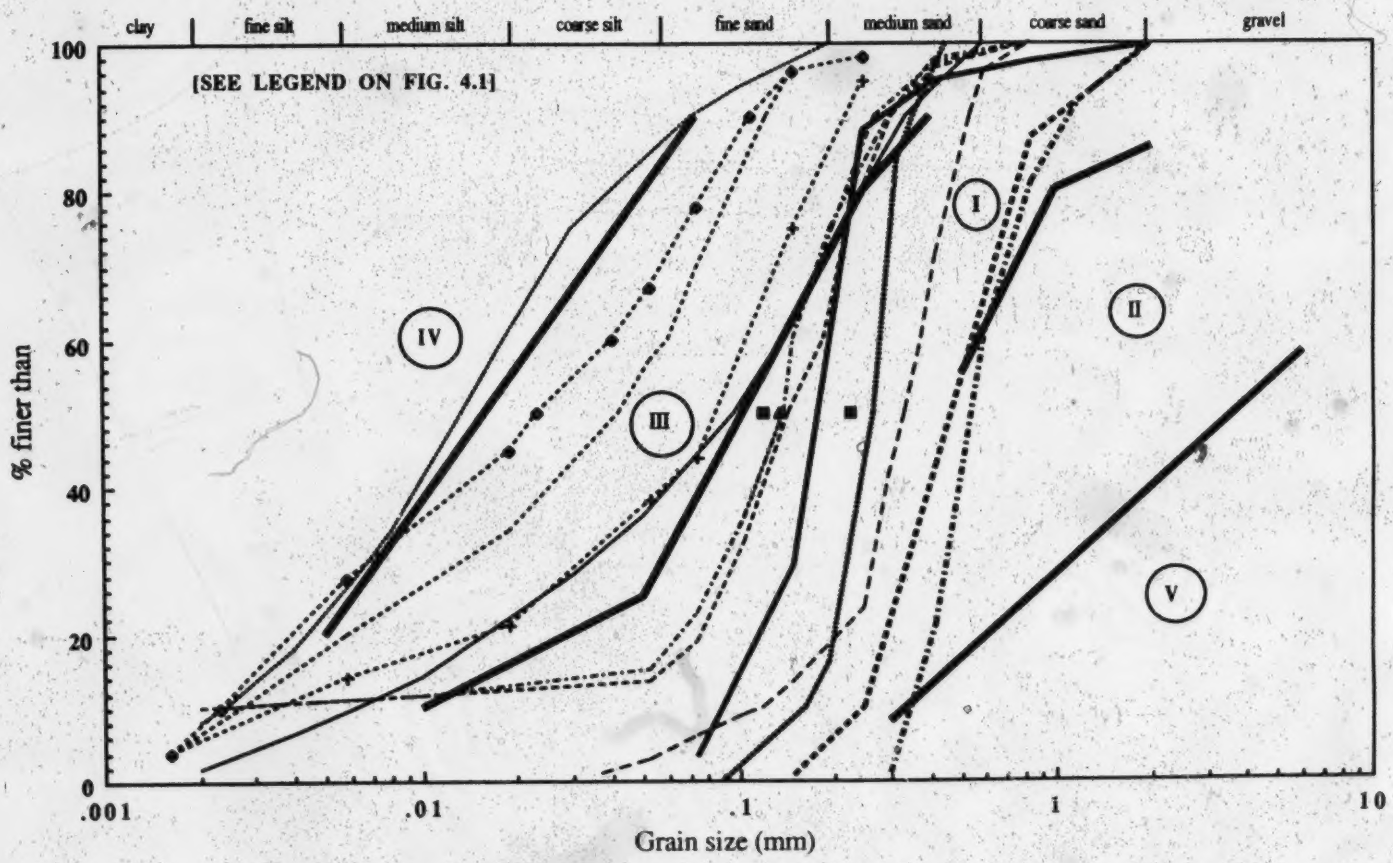

FIG. 4.2 - Grain size distribution curves of materials used in various flume tests compared to the Soviet criteria

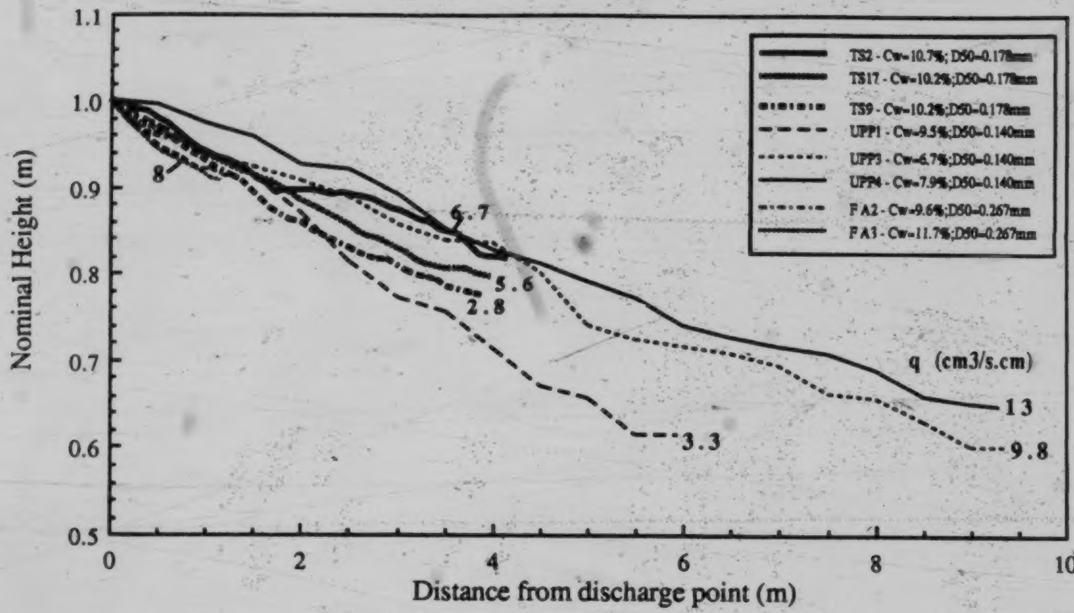

FIG. 4.3 - Profiles of TS, UPP and F tests for Cw=10% and various specific flow rates

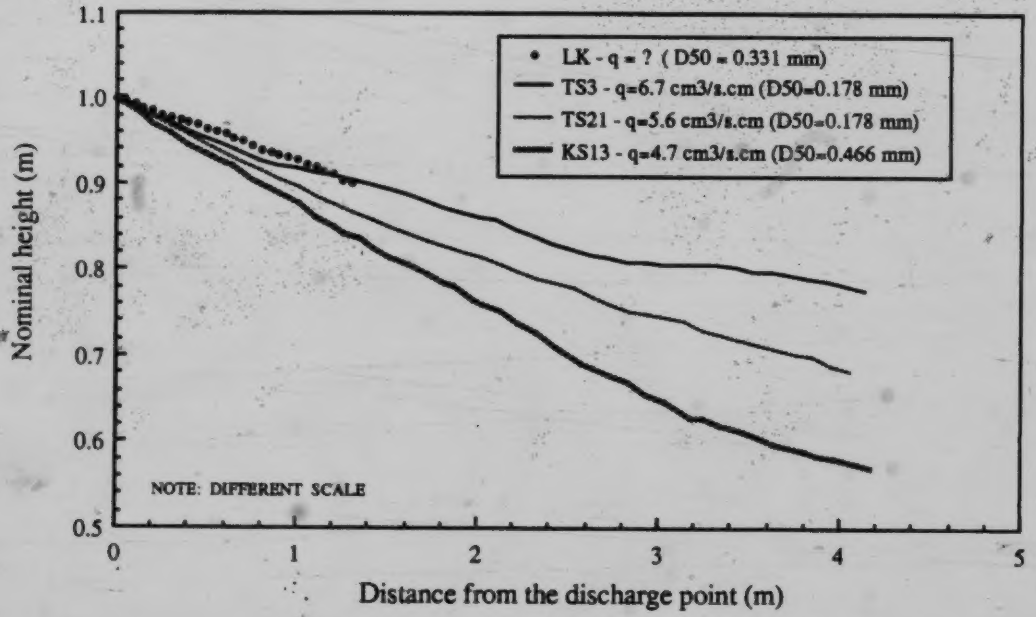

FIG. 4.4 - Profiles of TS, KS and LK tests for Cw = 20%

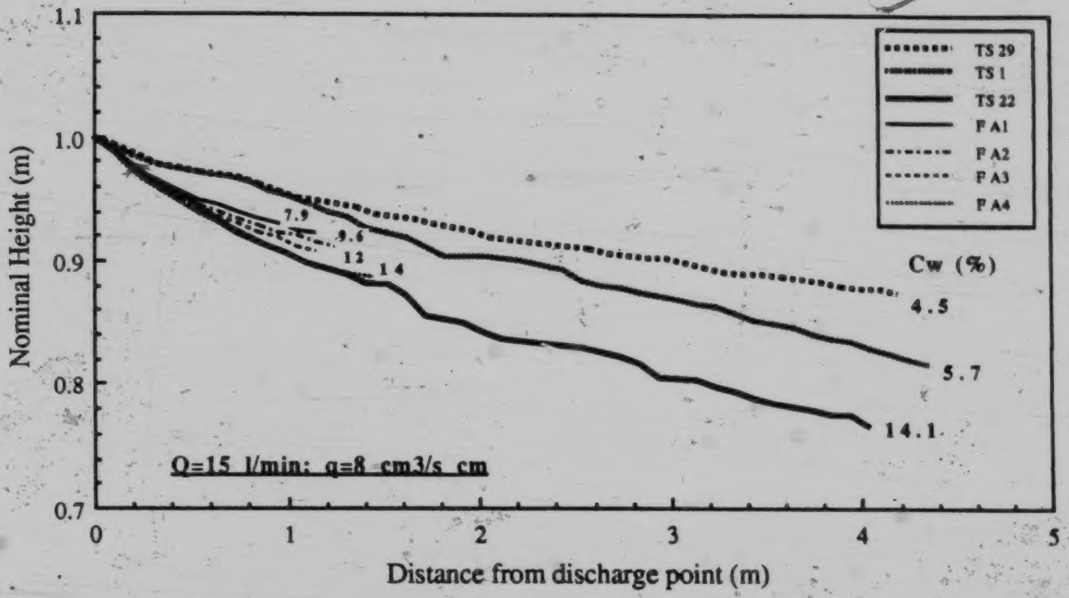

FIG. 4.5 - Comparison of profiles of TS (D50=0.178 mm) and F (D50=0.267 mm) tests

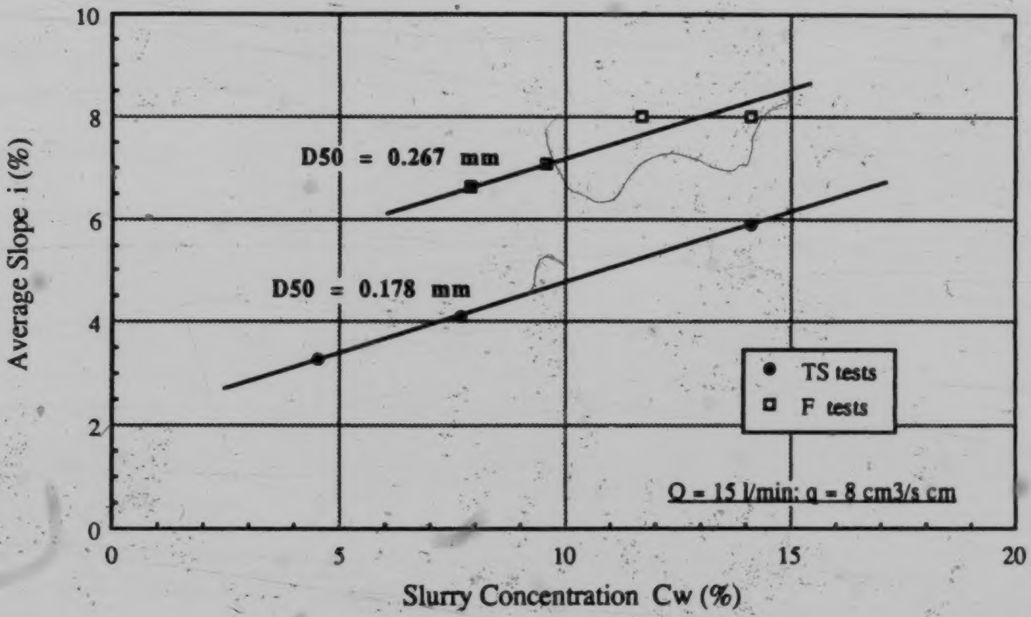

FIG. 4.6 - Effect of grain size and concentration on the slope

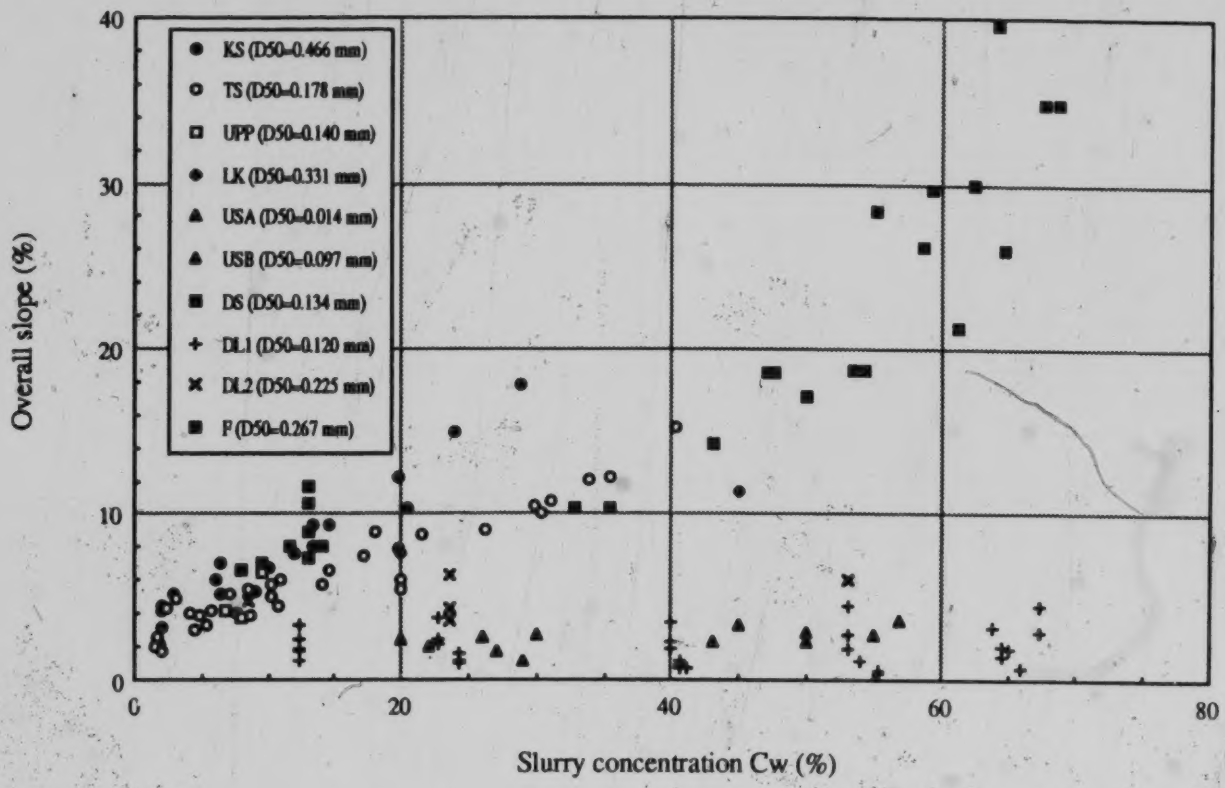

FIG. 4.7 - Variation of slope with slurry concentration for various flume tests (see Table 4.1)

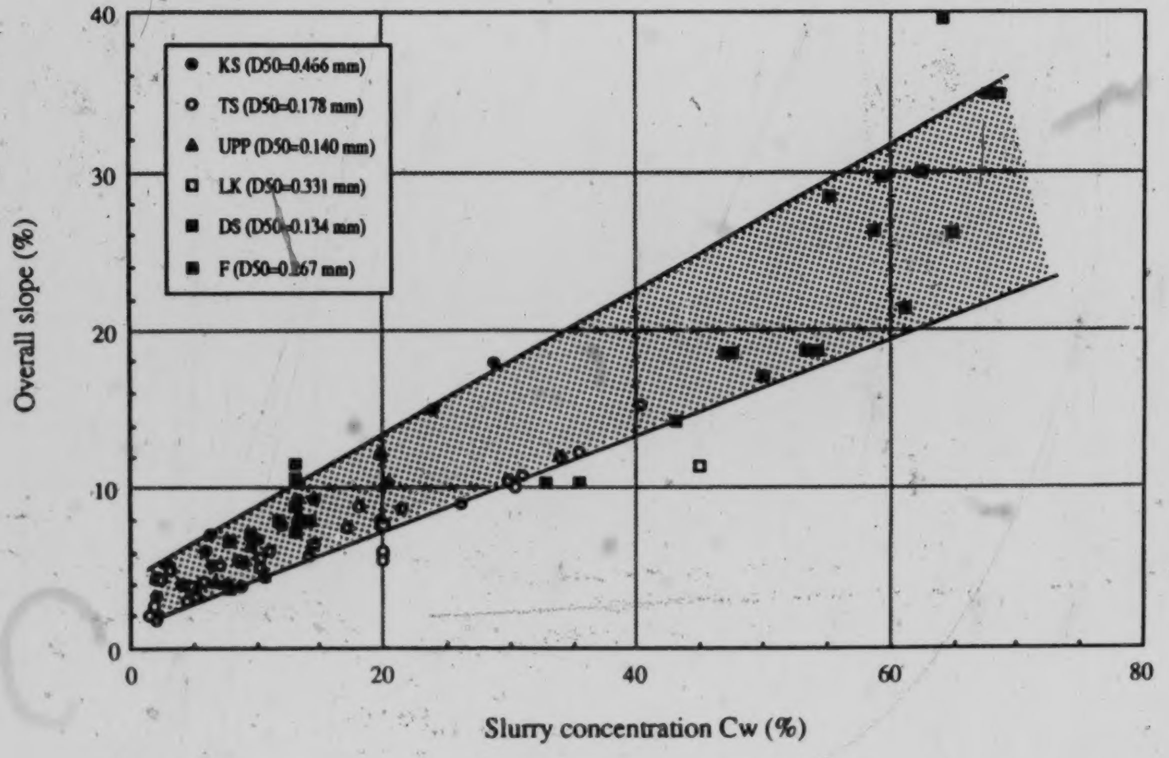

FIG. 4.8 - Variation of slope with slurry concentration for sandy fills with Q < 2500 cm3/s

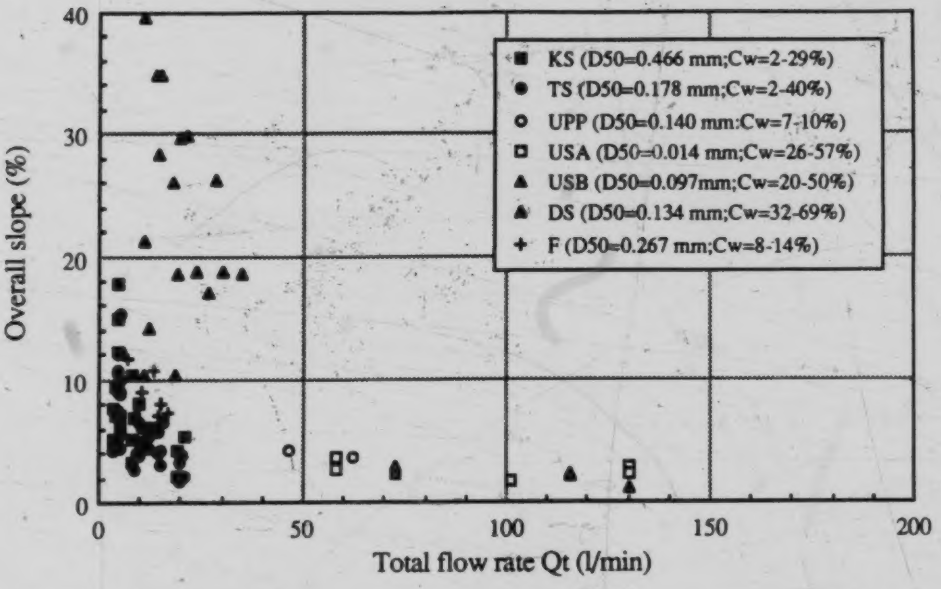

FIG. 4.9 - Variation of slopes with flow rate for various flume tests

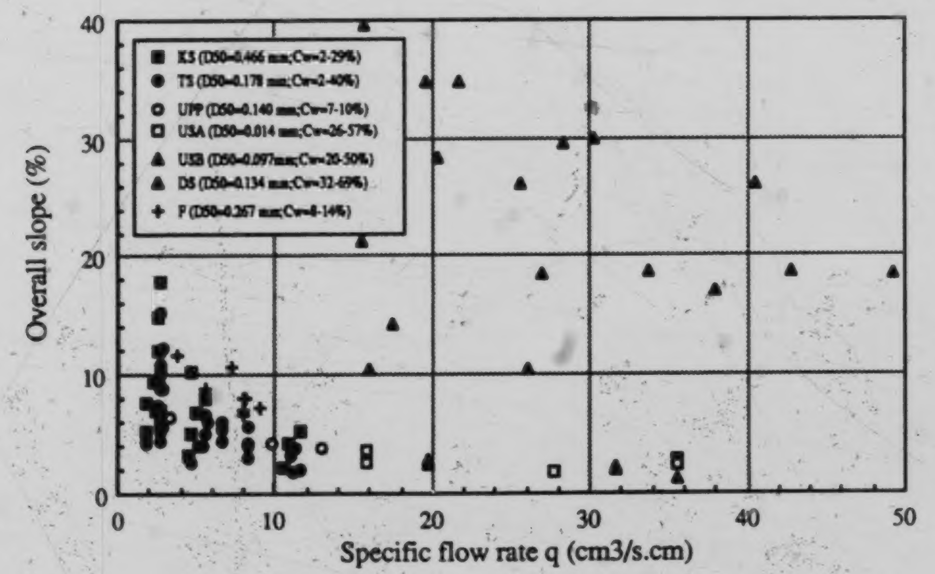

FIG. 4.10 - Variation of slope with specific flow rate for various flume tests

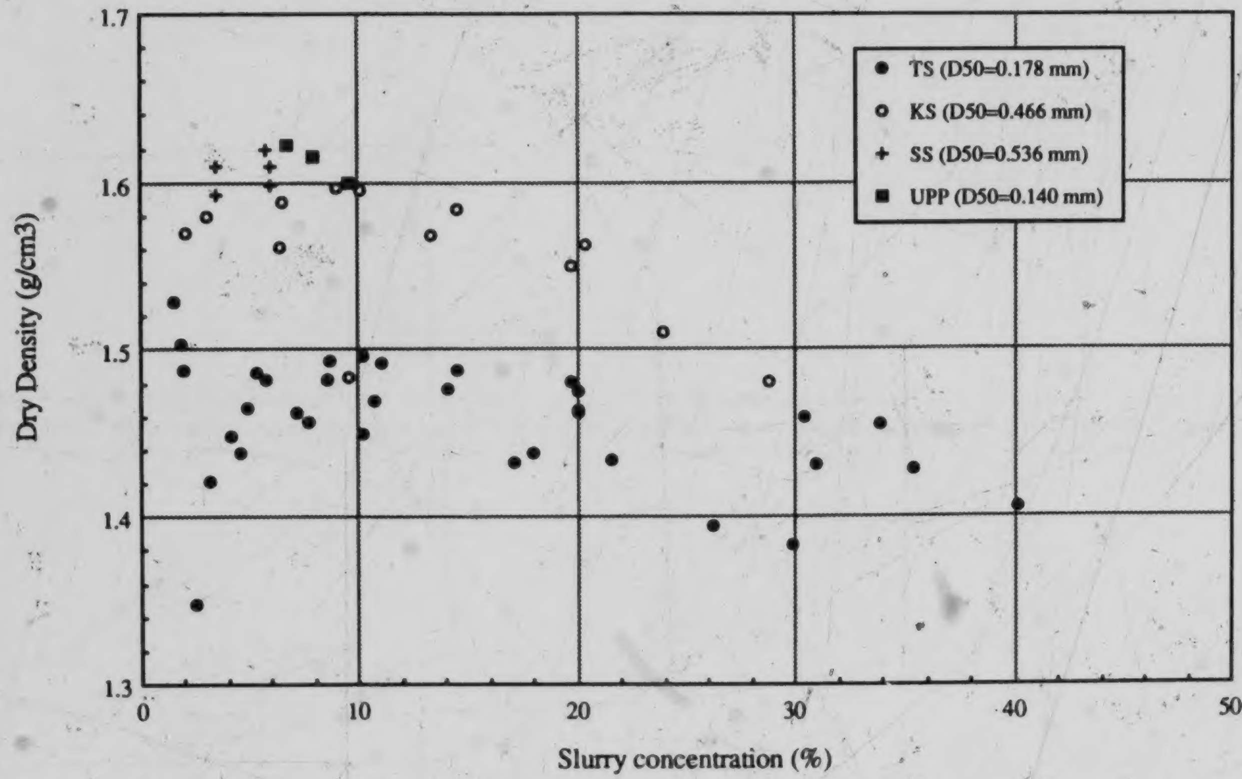

FIG. 4.11 - Variation of density of the fill with slurry concentration for various flume tests

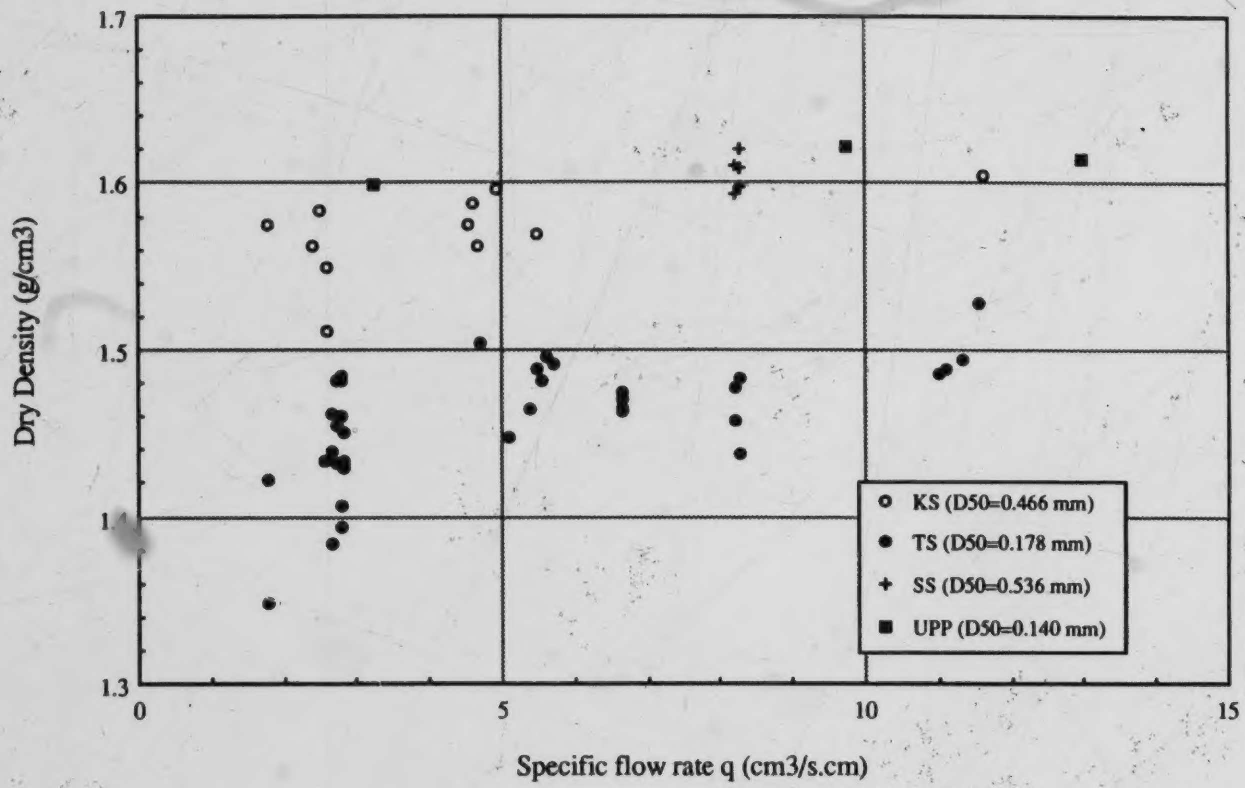

FIG. 4.12 - Variation of density of the fill with specific flow rate for various flume tests

Chapter 5

Field Deposition Tests to Study Hydraulic Fills

5.1 - INTRODUCTION

Hydraulic deposition is a practical and economical method for fill construction, that is particularly advantageous when the material to be deposited is already in a slurry form, such as dredged materials and tailings from mining and industrial operations.

The successful application of hydraulic deposition depends in many instances on the properties of the deposit being formed. If the fill shows unacceptable behaviour, extra effort and cost are involved in order to deal with problems such as static and dynamic stability, drainage and excessive volume and area occupied. The importance of understanding and controlling the fill properties is evident.

The properties of hydraulic fills depend on the depositional parameters such as slurry concentration, flow rate, grain size distribution of the solid fraction and on the construction method. Some of these depositional parameters were isolated and studied in laboratory flume tests described in Chapter 3. The results of these tests compare well with several other flume tests found in the literature (Chapter 4). However, because of problems associated with conventional hydraulic modelling of sediment transport phenomena (see Chapter 3), a similarity-of-process approach was adopted (Hooke, 1968). In this approach, laboratory tests are taken as fundamental tests, i.e., independent systems (not scaled versions of any prototype) which reproduce some of the characteristics and processes of the systems to be studied. Therefore, results obtained in the laboratory cannot be directly transferred to the field scale and conclusions drawn from the laboratory experiments require field validation.

Field tests were then carried out to check the validity of the laboratory findings discussed in Chapter 3 and 4 and to investigate the hydraulic deposition phenomena in a more general way. With these objectives in mind, eight large scale field tests were performed on a tailings dam. This chapter describes these tests in detail and summarizes the results. The comparison between the results of the laboratory flume tests and the field tests is discussed in Chapters 6 and 7.

5.2 - TEST SITE AND EXPERIMENTAL PROGRAM

The field tests were conducted on the tailings dam of Syncrude Canada Ltd., located 40 km north of Fort McMurray in Northeastern Alberta (Figure 5.1a).

The tailings dam is a ring dam with an approximate perimeter of 18 km and ranging in height from 32 to 90 m upon completion (Fair and Handford, 1986). For planning and construction purposes, the dyke perimeter is divided into segments called cells (Figure 5.1b). The dam is built using a modified centreline method, resulting in the typical cross section shown in Figure 5.1(c). The compacted cell is constructed by sluicing the tailings stream into construction cells which are surrounded by small dykes. Wide pad dozers are used to spread and compact the material during the sluicing process. In winter months when cell construction is not possible, the tailings stream is discharged from the dam crest to form a sandy beach. Most of the water and fines flow to the pond that is formed in the centre of the dam.

The field tests consisted basically of discharging the tailings stream under controlled conditions and monitoring beach formation. The fact that these tests were performed in an on-going large scale mine imposed a few limitations to what could be done and when it could be done, but had the advantage of working under actual operational conditions. The experimental program consisted of eight tests, a pilot test (Test 0) carried out during the Winter of 1986 and seven subsequent tests (Tests 1 to 7) conducted during 1988.

The pilot test was performed using the standard technique adopted by Syncrude for beach deposition by spigotting. The pilot test was an important preliminary step for the subsequent tests. The objectives of the pilot test were:

a) to provide a better understanding of the deposition phenomena and to provide a basis for comparison for other tests.

b) to provide background information for the design of subsequent tests in terms of equipment selection and operation, field measurements and expected quality and variability of results.

c) to provide reference data for the design of the laboratory tests and for the verification of the capability of the laboratory tests to reproduce the physical phenomena observed in the field.

The subsequent seven tests had more specific objectives. They were designed to study two variables of the deposition process: slurry flow rate and slurry concentration. Tests 1 to 4 had the main objective of studying the effect of the slurry concentration, while tests 5 to 7 were concerned primarily with the study of the effect of the flow rate. Table 5.1 summarizes these field tests.

5.3 - EQUIPMENT AND TEST PROCEDURE

All the field tests consisted of depositing slurry onto the beach from spigots installed on a main pipeline laid along the dam crest, according to the lay-out shown in Figure 5.2.

5.3.1 - Description of the equipment

Most of the tests used 5 spigots with diameter of either 7.6 or 15.2 cm (3 or 6 inches) installed on a pipeline of diameter 61 cm (24 inches) (see Table 5.1). The 7.6-cm spigots were 4.6 m long and the 15.2-cm spigots were 3 m long. Four of the five spigots were grouped (equally spaced every 24.4 m) to create a condition of two dimensional deposition where flow from different spigots could interfere with each other. The fifth spigot was placed at a distance (73.2 m) from the group to simulate a three-dimensional depositional pattern where the flow could form a fan. This last spigot was not used for the tests with the 7.6-cm spigots. The end of the pipeline was open, discharging the remaining slurry to the beach (called "overboard").

Special spigots were built for these tests in a small section of the pipe (2.44 m long 'spool-piece') that could be attached between regular sections of the pipeline by means of a special connector. This connector allows the rotation of the small section containing the special spigot to the desired position before securing the section in place. These special spigots were equipped with two valves that allowed a certain control of the flow and a sampling nozzle with its own valve for slurry sampling.

Flow rate was varied by the use of two different spigot diameters. The variation of slurry concentration was obtained by taking advantage of the fact that the flow in the pipeline is stratified. More solids travel along the bottom of the pipe, while a thinner slurry flows closer to the top of the pipe. Therefore, the highest slurry concentrations were obtained by installing the spigots so that the flow came vertically out of the bottom of the pipeline. For the lowest concentrations, the spigots were installed vertically on the top of the pipe. In both cases, an elbow was adapted to each spigot to have the flow discharged horizontally to the beach, so the angle of the jet was not changed. An intermediate value of slurry concentration was obtained by installing the spigots horizontally coming out of the side of the pipe. These different positions of the spigots are sketched in Figure 5.3 along with the standard position used at Syncrude and adopted for the pilot test (that is the spigot coming tangentially out of the bottom of the pipeline).

5.3.2 - Test Procedure

Prior to the beginning of the test the pipeline was laid along the dam crest and the spigots were installed on the test area and adjusted to their specified position. Graduated steel stakes were driven on the beach forming a grid in front of the spigots. Just before the beginning of the test, the area was surveyed to obtain the initial beach geometry and the exact position of spigots and graduated stakes.

The slurry discharge was then initiated and the flow velocity in each spigot was measured using an ultrasonic velocity meter. The flow in each spigot was adjusted by opening or closing the valves installed in the spigot in order to obtain similar flow rates in all spigots. The average flow rate in the spigots depended on the number, position and diameter of the spigots as well as on the number of variable speed pumps operating on the system and their settings.

The beach elevation at the graduated stakes and the flow velocity in each spigot were recorded several times during the tests. Slurry samples were also taken regularly during the tests at the sampling nozzle installed on each spigot and at a sampling point installed on the main pipeline immediately after the last booster pump house (called "feed" samples). At this position the turbulence in the pipeline was considered to be sufficient to yield a representative sample. After flushing the sampling nozzle, slurry samples were collected (in 125 ml jars) and sent to the laboratory for determination of slurry composition. It was not always possible to obtain slurry samples either because of sampling nozzles becoming frozen or because of unsafe access to the sampling point as a result of collapse of the dyke around the spigot. This collapse was usually caused by erosion due to the wind spraying the slurry jet against the dyke. After enough material had been deposited (controlled by the graduated stakes), the discharge was shut down and the area was allowed to drain. Depending on the particular conditions, the formed beach may have terminated upslope of the pond or may have degraded into the pond.

5.3.3 - Sampling and in-situ measurements

Undisturbed and remolded samples were taken from various locations along the newly formed beaches. Undisturbed samples were obtained either by drilling the frozen beach with a 4 inch hand-barrel or by statically pushing down 4-inch sampling tubes, excavating around them and freezing the material from the bottom upward with carbon dioxide pellets (dry ice). Thin walled PVC samplers produced more uniform freezing with less heave on the samples than metal samplers. The estimated flow direction was marked on each sample. This information was used for the fabric analysis, as described in Chapter 6.

Density and moisture content were determined in-situ using a double probe nuclear strata gauge (Strata Gauge model 501-DR by CPN Corporation). The probes contain a gamma ray source and a detector for density measurement and a fast neutron source and a thermal neutron detector for moisture determination. The strata gauge provides a profile of densities and moisture content values with measurements effectuated every 5 cm. The nuclear strata gauge was carefully calibrated in a large box in the laboratory using bulk samples taken from the test fill area. The field results obtained with this equipment are summarized in Appendix C. At most locations, undisturbed samples were taken at the same location of a strata gauge measurement in order to allow comparison of the results (see Appendix C and Section 5.5.5).

The undisturbed samples were kept frozen in large insulated boxes containing dry ice and were shipped to the laboratory at the University of Alberta in Edmonton, where they were trimmed in a -25° C cold room to remove the outer parts considered disturbed (see Chapter 3). These samples were used for determination of density and moisture content and for a fabric study (see Chapter 6). They were also used for triaxial tests by Law (1991) in a companion research work that investigated the mechanical behaviour of this material. Remolded samples from the test areas were also taken to the laboratory for grain size distribution analysis.

Surveys were carried out again after each test to determine the final geometry of the test area and to locate strata gauge and sampling points.

5.4 - FIELD TESTS RESULTS

The detailed results of the field experiments were reported by Küpper (1987; 1989a,b,c and d). A summary of these results will be presented in this section along with the description of relevant details of individual tests.

5.4.1-Pilot Test (Field Test 0)

The pilot test (Test 0) was performed on Cell 11 on the northwest side of the dam (Figure 5.1b) between December 2nd and December 9th, 1986. Sampling and post-test surveys were carried out during the period December 15-19, after the fill had drained sufficiently to allow access. The test was set up on Line 5 with 6 spigots installed horizontally off the bottom of the pipeline according to the standard set-up used at Syncrude (Figure 5.3). For this test, each spigot discharged slurry until the newly formed beach raised high enough to touch the base of the spigot (*h=0*, Figure 5.2). The original height *h* between the spigot and the beach varied between 3.8 and 4.5 m for this test.

Six graduated stakes were installed on the beach to monitor its variation in elevation during the test. However, graduated stakes 1, 2 and 4 were washed out at the beginning of the test and stake 5 was lost on the third day. During the test, measurements of the beach height below the spigots and beach elevation at the graduated stakes were obtained.

Determination of flow rate or slurry composition by direct sampling of the flow coming out of the spigot did not prove feasible. As an alternative, the trajectory of the slurry jet was determined and the order of magnitude of flow rate and velocity were estimated by applying Bernoulli's equation (Streeter, 1956). The mean exit velocity was estimated as approximately 5.5 m/s and flow rate in the order of 100 l/s.

The beach geometry obtained from level surveys carried out before and after the test are presented in Figures 5.4 and 5.5 respectively.The dam crest and the pipeline with the spigots are located on the bottom left of these figures. After discharge, the slurry flows towards the top part of these plots in the direction of the pond. Figure 5.6 shows the location of the longitudinal and transverse sections considered for the analysis as well as the sampling points. The beach profiles on these sections before and after the test are presented in Figures 5.7 to 5.14. Data for intermediate profiles were taken from graduated stakes, and therefore are not as accurate as the initial and final profiles obtained by surveying methods. Figure 5.14 is a transverse profile across the fill parallel to the dam crest (see Figure 5.6). It shows the thickness of the deposit decreasing along the pipeline from spigot 1 (upstream) to spigot 6 (the last one downstream). This preferential deposition is more accentuated at earlier stages of the test and implies that spigot 1 possibly draws a higher flow rate and/or a denser slurry.

Grain size distributions were determined for samples taken along the test fill and are summarized in Table 5.2 and densities measured on undisturbed samples are presented in Table 5.3.

5.4.2 - Field Test 1

Test 1 was located in Cell 6 (see Figure 5.1b). A special branch line was installed southwards from Cell 7, off the existing tailings line nº 5. A special switch was installed to allow tailings slurry to be diverted from the cell construction area in Cell 9 to the test area in Cell 6. It started on August 22nd and was shut down on August 24th, 1988 after 32.5 operating hours.

The input parameters for Test 1 are summarized in Table 5.4, which presents measured flow velocities, concentration and composition of the slurry at different times during the test. The velocity of the flow out of the spigots is on average 3.3 m/s which corresponds to a flow rate of approximately 60 l/s. Figure 5.15 shows the variation of slurry concentration with time. The overall average slurry concentration for all spigots is 64% with a coefficient of variation of 6%. The average value of slurry concentration on the spigots is higher than the feed concentration because the flow in the pipeline is stratified at the test section and the spigots were installed vertically out of the bottom of the pipeline. For the same reason, the solids coming out of the spigots are coarser and contain less fines than the feed material (Table 5.4). Slurry concentration decreases from spigot 1 (upstream) to spigot 5 (downstream end). The solid fraction also becomes finer towards the downstream. Spigot 3 has a slightly higher slurry concentration and lower percentage of fines relatively to its position along the line. This could be explained by the sagging of the pipeline at the location of spigot 3, that was consequently at a lower elevation than the others. The bitumen contents of the various spigots are very similar, around 0.17%, below the average bitumen content of the feed (0.28%), which is expected since the bitumen tends to float.

Test 1 started August 22 building up rapidly. For the remaining two days of this test, the Plant underwent a partial shut-down and the feed concentration dropped (Table 5.4). The test deposit adjusted to the new conditions by eroding the beach and forming temporary shallow channels. After the problem was solved, the beach was built back up as the concentration increased towards the end of the test. During the last day, spigots 1 and 2 were buried but still ran at reduced rates. This contributed to a better beach build-up in front of spigots 3 to 5, resulting in a more uniform beach. For most of the time the flow was well distributed across the entire surface of the beach, creating a steep uniform beach that drained and gained strength rapidly.

Figures 5.16 and 5.17 present the contour lines based on level surveys carried out before and after the test, respectively. The dam crest and the pipeline are shown on the upper left side of these figures. The slurry flows on the beach to the right towards the pond (not shown). The beach surface is relatively smooth with contour lines parallel to the dam crest. Figure 5.17 shows deposition of new material in front of the group of spigots 1 to 4 and isolated spigot 5 as opposed to almost no new material in between. The location of the cross sections used for analysis, the position of the graduated stakes and the positions of the samples taken during the field investigation are indicated in Figure 5.18.

Figures 5.19 to 5.26 present the beach profiles before and after the test for each of the cross sections shown in Figures 5.16 to 5.18. The location of the undisturbed samples are also shown on these profiles. Test 1 was started with the spigots at approximately 4 m above the beach elevation and was shut down when the deposit reached the spigots level.

Grain size distribution analyses were performed on remolded samples collected across the test area and a summary of the results is shown in Table 5.5. Moisture content and density were determined from frozen undisturbed samples. The average values are summarized in Table 5.6. In some locations (such as SG 1,3,4,17,19 and 20) more than one sample was taken on the same vertical. They are indicated in Table 5.6 by additional digits on the sample number, 1 being the most superficial sample and 3 being the deepest one. In all cases, samples from areas where no deposition occurred were disregarded when calculating average values.

5.4.3 - Field test 2

The spigots used for Test 1 were dismantled and re-assembled further south in Cell 6 on the special branch line off tailings line 5, but this time the spool pieces were rotated so that the flow was taken off the centre of the pipeline. Test 2 started September 15th, 1988 and was shut down on September 30th, after 254 hours of operation.

This test had such an extended duration because the deposition was very unstable, depositing and eroding in a cyclic pattern. A typical cycle would start with the jet from each spigot cutting a sinuous channel with regularly spaced meanders, several meters deep at the top of the beach and shallower further downstream. The flow in these channels erodes the banks increasing the sinuosity and introducing extra sediment load in the stream. The channels were gradually infilled and broadened, reaching almost the original beach level in some cases. Then the flow would start cutting the channel again, re-initiating the process. This phenomenon seemed to have cycles of approximately 2 hours and it did not seem to happen necessarily in all channels at the same time. For most of the 15 days of this test there was no permanent variation in beach elevation, with all the material discharged being transported to the pond.

The input parameters for Test 2 are presented on Table 5.7 and Figure 5.27. This graph does not seem to show any obvious cyclic variation of these parameters that could explain the deposition pattern, although some observations of severe channel erosion do coincide with low values of slurry concentration.

Figures 5.28 and 5.29 present the contour lines obtained from surveys carried out before and after the test, respectively. The shaded areas in Figure 5.29 represent the simplified outline of the bottom of the channels. The average height from the dam crest to the beach was around 5 m at the beginning of Test 2. At the end of the test it had reduced to 3 m in areas that had no channels. The height between the dam crest and the bottom of the channel in front of spigot 1 was over 6 m at the end of the test and it was 5.5 m for the channel in front of spigot 5. The other channels were even deeper, but because they had water on the bottom and the edges were unstable it was not possible to determine the exact height. The cross-sections analyzed and the location of the undisturbed samples and strata gauge measurements are shown in Figures 5.29 and 5.30. The beach profiles along these cross sections are presented in Figures 5.31 to 5.36. Profiles I and V (Figures 5.31 and 5.35) show that no net deposition occurred in the channels, except far from the discharge points where the channel was already very shallow (area of samples SG6 and SG7 on profile I, for example).

The laboratory tests performed on Test 2 samples consisted of grain size distribution analysis summarized in Table 5.8 and moisture and density of undisturbed samples (Table 5.9).

5.4.4 - Field Tests 3 and 4

The same spigots used for Tests 1 and 2 were installed on the main line (Line 5) in Cell 8, but rotated so that the flow came out of the top of the pipe for minimum slurry concentration. Test 3 started on October 23rd, 1988 and was terminated prematurely on October 29th due to a communications problem between operation and research. There were no channels, with uniform flat beaches formed by sheet flow coming out of plunge pools formed by the slurry jets hitting the beach. The rate of build up as determined by the graduated stakes was low. Cell construction could be carried out simultaneously as the spigots on the top of the pipeline were only removing a relatively thin slurry containing a lot of fines. As reported by the dozer operators this process was even advantageous for cell construction. Since the test spigots were installed upstream of the construction point and were removing a relatively large amount of fluid and fines, a thicker slurry (high slurry concentration, coarser material) was reaching the construction cell. This caused an increase in the rate of construction and a reduction in the size of the discharge hole, creating a firmer deposit where dozers move easily and safely. It also required less track packing to dewater the fill. The reduced flow rate and reduced water content of the slurry also minimize the wash out of the dry dykes and the tendency to cut channels. However, the installation of spigots on top of the pipe seems to aggravate the problem of dyke erosion at the spigots site by wind spraying the jet towards the dam crest. Although the test ran for 6 days, it could not be considered complete due to the low rate of deposition. Slurry samples were taken but the beach did not build up enough to be sampled. The data obtained are presented in Table 5.10.

Test 4 was a repeat of Test 3 that was incomplete and hence had exactly the same set-up. It was installed on the main line (Line 5) on Cell 7 a few hundred meters south of the Test 3 area. The test was carried out for almost 8 days, this time 24 hours a day and with no concurrent cell construction. The average temperature during Test 4 was -15° C, causing several problems of sampling nozzles freezing up. Even lower temperatures had contributed to problems also on the installation of the spigots, which was probably the cause of a partial collapse of spigots 2 and 4 during the test. Despite these difficulties, Test 4 was considered successful and yielded consistent results.

Table 5.11 presents Test 4 input parameters, which varied in time as shown in Figure 5.37. The beach geometry is shown by the contour lines before (Figure 5.38) and after the test (Figure 5.39) and by the profiles through cross-sections I to IV (Figures 5.40

to 5.44). The summary of the grain size distribution analysis are presented in Table 5.12 and the results of moisture content and density determinations are in Table 5.13.

5.4.5 - Field Test 5

This was the first test with the 7.6-cm spigots. Instead of being built in individual spool-pieces, nine of these small spigots were installed in a single 24.4 meter long section of pipe (diameter 61 cm). The spigots were 4.6 m long and were 2.45 m apart. For Test 5 only 4 spigots were used, every second one being shut. The distance between operating spigots was then 4.9 m. The spigots were installed coming out of the centre side of the pipeline on Line 3, Cell 30 (Figure 5.1 and 5.3c). This test was not successful due to installation problems, initially due to a worn out orifice plate at the end of the pipeline and later due to extreme erosion of the dyke at the test section.

5.4.6 - Field Tests 6 and 7

Test 6 was a repetition of Test 5 with 7.6-cm spigots coming out of the centre of the pipe, but installed on Line 1 at Cell 18 on the north side of the dam. This test started on November 24th, 1988 and ran for 11 days (94 on-line hours). Very low temperatures during Test 6 caused one of the spigots to freeze up and impaired the proper functioning of the ultrasonic velocity meter. Flow rates were estimated as being in the range of 15 to 20 l/s in each spigot, corresponding to a velocity of approximately 3 to 4 m/s. The beach built up rapidly at first but then a channel was eroded. It seemed that the extent and the severity of channelling was reduced due to the low flow rate. The channel deviated to the side cutting through the area where Test 7 was to be performed, instead of cutting through the previously deposited beach. The slurry composition during Test 6 is presented in Table 5.14 and Figure 5.45.

Test 7 was carried out beside Test 6 area on Cell 18. It had a similar lay-out, except that the spigot section was rotated to have the spigots coming out of the bottom of the pipeline. Instead of becoming horizontal after the off-take, the spigots were installed facing downwards some 30° below the horizontal, in an attempt to prevent freezing by increasing the flow velocity. Test 7 was carried out from December 7th to December 12th, 1988. The average temperature was -22° C and only 2 spigots performed satisfactorily throughout the entire test. Table 5.15 presents the slurry composition data for Test 7.

Figure 5.46 shows the geometry of the area where Test 6 was to be performed. The flow on the beach is from the top of the figure, where the crest is located, to the bottom towards the pond. Figure 5.47 presents the contours of the Test 7 area before the test was carried out, based on a survey done after Test 6 was performed adjacent to it. The shaded area corresponds to a channel cut by Test 6 flow. The combined after test survey results for Tests 6 and 7 are presented in Figure 5.48. The effect of the channel cut by Test 6 is noticeable on the final geometry of Test 7. The profiles through the cross sections indicated in Figures 5.46 to 5.48 are presented in Figures 5.49 to 5.52. It is possible to observe in these profiles the control of the previous surface over the new profile over the areas where there was not enough deposition to fully develop the new surface. Unfortunately these test areas could not be sampled.

5.5 - DISCUSSION OF THE EXPERIMENTAL RESULTS

An interpretative summary of all field tests results is presented in Tables 5.16 to 5.22. In these tables, values marked with an asterisks were not considered for calculation of the average and the average values of the sample characteristics are averages of all samples and not the average of the partial averages listed above each one. Specific items in these tables will be explained and discussed in the following sections.

5.5.1 - Input Parameters

The stratification of the flow in the pipeline was demonstrated by the different slurry concentrations obtained by varying the position of the spigots around the circumference of the pipe. When the spigots were installed on the top of the pipe (Tests 3 and 4), the slurry concentration obtained was approximately 40%. The average concentration increased to 55% for cases where the spigots were coming out of the centre side of the pipe (Tests 2, 5 and 6) and to 65% when the spigots were rotated to the bottom of the pipeline as in Tests 1 and 7.

The stratification of the flow in the pipeline also produces fines segregation. This is caused by the fact that the settlement of solids in the pipe occurs when the velocity is smaller than the critical velocity. The finer the particle, the lower is the critical velocity. In this case, the finer fraction of the slurry takes a very long time to settle even in stationary conditions. For this reason, the top layer of the flow in the pipeline contains less total solids but relatively more fines than lower layers. Therefore, spigots installed on the top of the pipe drew a slurry that contained more fines than the slurry flowing through the spigots installed on the bottom of the pipeline. For example, the spigots of Test 4 (on the top) yielded an average amount of fines of 42% while Test 1 (spigots on the bottom) had an average of 11% fines in samples from the spigots. Test 4 was fed a slurry with more fines (≈ 29%) than Test 1 (19%), but this is not as significant as the difference in the amount of fines through the spigots due to their position around the pipe. Test 2 (spigots on the side) and Test 4 (spigots on the top) were fed slurries with similar composition and the amount of fines in Test 2 spigots (32%) was significantly lower than for Test 4 (47%).

Tables 5.17 to 5.20 present the relation between the amount of fines in the spigots samples ($\%F_{sp}$) and the amount of fines in the feed samples ($\%F_f$) for the various spigot positions. Also shown is the slurry concentration (refers to the amount of total solids) in the spigots ($C_{w/sp}$) in relation to the slurry concentration in the pipeline ($C_{w/f}$). Comparing both parameters it is possible to conclude that the segregation of fines is even more accentuated than the segregation of total solids.

The bitumen in the slurry tends to float and therefore segregates in a similar way as the fines. Consequently spigots installed on the top of the pipe deliver a higher percentage of bitumen compared to the total amount of bitumen in the pipeline than do the spigots installed at lower positions.

The impact of the segregation of fines in the pipeline on the test program was that two parameters were varied simultaneously, slurry concentration and amount of fines, complicating the discussion of the results. In fact, a third parameter was also varied concurrently: the height *H* between the spigots and the beach surface. Test 4 beach was more than 8 m below the dam crest at the end of the test, while the beaches of Test 0 and Test 1 were at the crest level at the end of these tests. The remainder of these tests had intermediate values of *H*. The effects of the height *H* between the spigots and the beach surface on the fill characteristics are not yet clear, but both Soviet and Chinese techniques call for minimization of the height *H* by placing the discharge point right on the beach surface. The techniques used in these countries also include the placement of discharge points facing upwards to minimize the discharge velocity. In all cases here except Test 7, the spigots were horizontal. For Test 7 the spigots were installed facing downwards to increase velocities in an attempt to avoid freezing during the test. Small diameter (7.6 cm) spigots, as used for Tests 5 to 7, do not operate well under winter conditions (temperatures between -10 °C and -40 °C); 15.2-cm spigots have a better performance.

Concluding, the rotation of the spigots proved to be an easy and economical way of varying the slurry concentration, although the range of concentrations that can be obtained is limited to the range of concentration occurring in the pipeline. A disadvantage of this system is that it causes not only a variation in concentration but also causes a variation in slurry composition.

The analysis of the results was carried out considering for the input concentration both the concentration of total solids in the slurry (C_w) and the concentration of sand in the slurry ($C_{w(sand)}$). The use of the latter is based on the concept that the fines behave as part of the carrier fluid and do not contribute to the mechanisms responsible for the formation of the beach material. Fines were considered to be the fraction smaller than 74 µm although other sizes could also have been adopted. The use of concentration of sands in the slurry did not reveal any additional information in relation to the use of concentration of total solids. For this reason and because it includes an arbitrary factor, the definition of the size of fines that act as part of the carrier fluid, the concentration of sand in the slurry is not included in the following discussion of results.

5.5.2 - Beach flow and deposit

The behaviour of hydraulically placed soils depends primarily on its fabric and grain size distribution, which in turn depend on how the material was deposited. The mode of deposition is influenced by the composition of the slurry and the flow conditions. Therefore, in order to help understanding the formation of the fill and its behaviour, a detailed description of the flow conditions is presented.

The flow was discharged from each spigot at full section. The trajectory of the jet leaving the spigots was used in some cases for a rough estimate of flow rate, using Bernoulli's equation. Due to the high temperature of the slurry (≈ 70°C) much fog was formed on the colder days, making observation of the beach flow more difficult. In some cases, wind blowing from the pond area caused the jet to spray and erode the dyke, badly enough in certain cases to endanger the stability of the pipeline laid on the dyke. This phenomenon, of practical importance for hydraulic fill construction, was more critical for sites where the dyke was significantly higher than the beach, and seemed to be aggravated when the spigots were installed on the top of the pipeline, which causes the jet to be more exposed to wind.

On hitting the beach, the jet forms a plunge pool where oversized grains may accumulate. When the discharge is shut down, the plunge pool remains full of slurry, forming a soft spot in the beach. The plunge pool may overflow all around generating sheet flow in the immediate area or it may breach in one location forming a channel, usually in the direction of the momentum of the jet.

After leaving the plunge pool, the flow may open up forming a delta or concentrate in one or a few channels before entering the pool. When the flow forms a delta, it may present two types of drainage pattern that occur simultaneously: sheet flow and braided flow. In the field tests described above, sheet flow was localized and occurred as non-primary flow out of the erosion hole, over some bars and in localized areas of low flow velocity further downstream. Braided flow occurs extensively over beaches where large channels are not present. Coarse sediment tends to accumulate on top of bars rather than on the bottom of channels. In some cases, instead of forming a delta, the flow concentrates in channels. These can be shallow channels that soon are filled up and overflow, or deep channels that flow all the way to the pond discharging their sediment load under water. This latter type of channel is detrimental for fill construction. Although registered only in Test 2, the deep channelling phenomenon is common at Syncrude's tailings dyke, where is well known by the field operators, who do not need to relocate the pipeline when it occurs.

The channels create meanders, terraces, islands, etc. just like a miniature river system and should affect the characteristics of the material along the beach in the same way as a river influences the properties of its deposits. Lateral migration of the channels cause bank failures. These failures are mainly toppling failures of wedges that have already fractured and that are undermined by the flow. They provide substantial increase in the sediment load of the flow, since in most cases they are rapidly washed away. The cracking of these blocks expose fresh faces where the stratigraphy of the deposit can be observed. Inside the larger channels, the flow occasionally developed a braided or less often sheet flow pattern.

Some of the bitumen in the slurry is in the form of pellets and is deposited with the sand. The remainder is in liquid form; it floats and flows to the pond where it is collected by a plastic boom for recycling. A small amount of this liquid bitumen may end up on temporary or stagnant waters and may be deposited on the beach forming a dark sticky layer.

Regardless of drainage pattern, the most common bedforms observed in the field tests were antidunes in various stages. High energy antidunes in transition to chutes-and-pools, similar to the antidunes found in the Medano Creek, Colorado, USA (Langford and Bracken, 1987), were widespread on the beach, predominantly on the upper half. Relatively low energy antidunes in the form of trains of standing waves were observed on the lower part of the beach in the deeper flows of moderate velocity. Areas of secondary flow where the velocities were the lowest presented upper-stage plane beds with visible current lineations. Chutes-and-pools were formed in the regions of highest velocities, where there is flow concentration. Chutes-and-pools move slowly upstream, causing substantial remolding of the fill. Nick points were also common in the larger channels, remolding the beach to a considerable extent.

Some of the localized features of the flow include lag deposits which are composed mainly by particles that are larger than the average. These lag deposits will affect the extent and pattern of variability of the grain size distribution along the beach and consequently the other beach characteristics as well.

The laminated structure of the beach deposit can be observed on the banks of the channels. Layers vary in thickness from a few millimeters to a couple of centimeters and some layers can be traced for several meters along the bank. Cross-stratification and erosional features are also present.

The flow characteristics described above, especially drainage pattern and bedforms, are very important since they define the deposition process, and therefore the deposit properties.

5.5.3 - Grain size distribution

Syncrude's tailings sand is very uniform and hence hydraulic sorting was not expected to occur. According to the Soviet standard specifications for hydraulic fill construction (SNiP-II-53-73) sorting is not significant whenever the sand has $D_{60}/D_{10} \leq 2.5$ and/or $D_{90}/D_{10} < 5$. The solid fraction in Syncrude's tailings slurry is above these limits (D_{60}/D_{10} of 2.3 and D_{90}/D_{10} of 3.6) and accordingly, some hydraulic sorting was observed as shown in Figure 5.53.

The amount of fines in the beach material increased and the mean grain size decreased with distance from the discharge point. There was also a trend of increase in the coefficient of uniformity (D_{60}/D_{10}) with distance (Figure 5.53). The scatter of these data is in part due to the type of flow that deposits the beach, as discussed in the previous section. Layers or pockets of coarser material can be formed in areas that have been bars or lag deposits, while relatively finer material is found in areas that corresponded to the bottom of channels. Due to the nature of the flow on the beach, coarser and finer zones may alternate laterally or vertically as the flow varies in space and in time.

The tendency of the beach material to become finer and less uniform towards the downstream did not occur inside the deeper channels of Test 2, where exactly the opposite was observed (Figure 5.54). Inside the channels the mean grain size increased with distance from the deposition point and both the coefficient of uniformity and the amount of fines decreased.

5.5.4 - Fines capture

Fines capture refers to the deposition of fines in the voids of the sand matrix that constitute the beach material, as opposed to fines flowing to the pond and forming sludge. This concept is especially interesting for the cases of hydraulic deposition of materials that contain some sand and an appreciable amount of fines, which being drained away with the fluid, may cause an environmental and/or a materials handling problem. By capturing at least a portion of these fines in the voids of the deposited sand, it is possible to minimize potential problems and costs. However, the increased amount of fines in the beach sand may flatten the slope and may be detrimental to the geotechnical properties of the beach material. Therefore, the trade-offs of enhancing fines capture must be considered in the design.

Fines capture efficiency is defined here as the amount of fines in the beach material ($\%F_b$) in relation to the amount of fines discharge by the spigots ($\%F_{sp}$):

$$FC = \frac{\%F_b}{\%F_{sp}} \qquad (5.1)$$

This parameter gives a measure of the efficiency of the fines capture process by quantifying the proportion of the discharged fines that actually deposit on the beach. Deposition of slurries with more fines will obviously cause the beach material to have more fines. However, more fines also flow to the pond. By calculating the fines capture efficiency as defined above, it is possible to evaluate the ability of different deposition methods in minimizing the amount of fines that will form sludge, independent of the amount of fines in the slurry being discharged.

The fines capture parameters (*FC*) calculated for the various tests are shown in Tables 5.17 to 5.20 and Figure 5.55. The higher the slurry concentration, the more efficient the fines capture. There seems to be a sharp increase in fines capture efficiency above a certain value of slurry concentration around 60%. The fines capture efficiency inside the deep channels of Test 2 is very low, being less than half of the value obtained for other areas of the same test.

5.5.5 - Beach density

Deeper samples or samples from locations that did not receive substantial amount of new material during the test were not considered for the analysis of beach density. For this reason, the densities of these samples are not included in the average density calculated for each test.

Beach density was measured in-situ with a nuclear probe strata gauge and in the laboratory using the undisturbed samples that were frozen in the field. Two methods were utilized in the laboratory to determine density: the wax method (ASTM D1188-83) and direct determination by measuring regular shaped samples (see Appendix C). Densities obtained by the wax method and by direct measurements were consistent, but nuclear strata gauge results differed from both laboratory methods. Moreover, the discrepancy between strata gauge and laboratory measurements did not display a constant trend, being higher than the laboratory values for some tests and significant lower for others. Consequently, strata gauge results were not utilized in the analysis of the tests results. The values of density determined by the wax method were considered the most reliable ones and were the only values included in the analysis.

The variation of dry density along the beach for the various tests is shown in Figure 5.56. As for the flume tests, no consistent trends were observed.

Among the causes of the considerable scatter in the dry density data are difficulties and inaccuracies associated with the methods of determination of density as discussed in the Appendix C, and the nature of the flow that forms the fill. The hydraulic deposition process results in the formation of thin layers that can differ slightly from each other in composition and structure, and therefore in density. Some undisturbed samples of Test 2 (SG5, SG9, SG10, SG11 and SG22) were sliced and each piece had its density measured using the wax method; a significant difference in density was observed from slice to slice (see Table 5.9). The same variability that occurs vertically from one layer to the other can also be observed laterally within short distances, caused by the nature of the flow. This emphasizes once more the importance of understanding the flow conditions to understand beach characteristics and their variability. Localized features of the beach such as a plunge pool, a pre-existing channel or a volcano-like mound that can be formed around a spigot also account for points of low density in the beach.

The beach material becomes finer towards the downstream and for a particular material, the smaller the mean grain size, the lower the density. This is mainly due to the fact that finer particles tend to be less equant and more angular than larger grains and both factors contribute to a lower density. On the lower part of the beach, the flow velocity can be very low, facilitating sedimentation and consequently creating conditions to deposit material of relatively lower density.

Due to the lack of a definite trend of variation of density with distance from the discharge point, an average value of density for each cross-section will be adopted for all subsequent analysis.

The variation of beach density with the concentration of the slurry being discharged is shown in Figure 5.57. Since there was a variation in slurry concentration from spigot to spigot along the pipeline, the average density for the cross-sections located in front of each spigot were plotted against the slurry concentration of that particular spigot. The solid points in Figure 5.57 correspond to the overall average values of density and slurry concentration for each test. Slurry concentration is not the only variable in this case, which complicates the interpretation of this graph.

The slurry discharged in Test 4 contained much more fines (47%) than the slurry discharged in Test 1 (11%). Consequently, the beach material in Test 4 was also finer

(D_{50} = 0.137 mm) than the material deposited in Test 1 (D_{50} = 0.183 mm) which contributes to the low densities of Test 4 compared to Test 1. Also, for Test 4 the spigots were very high in relation to the beach level, starting at approximately 9.5 m and finishing at 8.2 m, while Test 1 spigots were at 4 m above the beach at the beginning of the test and were buried at the end. It is not very clear yet what are the effects of the height H between the spigots and the beach on the deposition process (see Item 5.5.1) but if this factor has any influence, it is likely to have affected the results as the two tests just mentioned present extremes values of H. Test 2 had a very irregular deposition and the factors affecting its beach density are even more complex. Test 0 had an average density of 1.54 g/cm^3, but the slurry concentration is not known. Test 0 spigots were installed using Syncrude's standard procedure (Figure 5.3a) and therefore the slurry concentration might have been in between the values obtained for Tests 1 and 2.

A detailed study of the maximum density (ρ_{max}) and minimum density (ρ_{min}) for the beach material was performed and is presented in Appendix D. It was found that both the maximum and minimum densities are sensitive to small variations in grain size distribution, confirming the findings of Burmister (1962), Kolbuszweski and Frederich (1963), Youd (1973) and Poulos and Hed (1973) among others. Different ways of selecting the maximum and minimum densities for the field samples were tried, including:

a) use of the average values of ρ_{max} and ρ_{min} determined for various field samples

b) use of ρ_{max} and ρ_{min} obtained from lines of constant C_U and constant D_{50} drawn through the points of ρ_{max} and ρ_{min} determined for various field samples

c) use of ρ_{max} and ρ_{min} obtained for a field sample that had values of C_U and D_{50} similar to the values of C_U and D_{50} of the sample being analyzed.

Although the values of ρ_{max} and ρ_{min} that were adopted may change the calculated relative density by as much as 25%, there is no difference in the relative effect of slurry concentration as shown in Figure 5.58. The relative densities obtained in these tests were in the range of 20 to 45%. The relative density for Test 0 is between 52 and 54%, significantly higher than the relative densities for the other tests.

5.5.6 - Beach geometry

The geometry of hydraulic fill beaches depends, among other factors, on the composition and concentration of the slurry being discharged. The average beach slope tends to increase for denser slurries and coarser materials. The results of the field tests were consistent with this, as the average beach slope increased with both the slurry concentration (Figure 5.59) and the mean grain size (Figure 5.60).

The slope inside the deep channels in Test 2 was relatively flat for the correspondent slurry concentration, as expected (Figure 5.59). Tests 6 and 7 operated with smaller flow rates than Tests 1 to 4, and therefore should have developed steeper slopes than Tests 1 to 4 for the same slurry concentrations, instead of similar slopes. It is possible that because of the relatively short duration and low flow rate, Tests 6 and 7 had not developed equilibrium profiles yet when the tests were terminated. In fact, Profile II, Test 7, for example, shows very clearly the control of the previous beach surface on the final shape of the test beach, after a point approximately 60 m from the discharge point. The average slope calculated for the first 60 m of this profile is 9%, which is more in the range of values that were expected for these tests.

The test set-up used for this program caused Test 1 to have the highest slurry concentration and the highest mean grain size, while the finest slurry was discharged at the lowest concentration during Test 4. Since an increase in either the slurry concentration or the mean grain size will create a steeper slope and a decrease in either of these parameters will flatten the slope, the variation of slope with C_w and D_{50} (Figures 5.59 and 5.60) becomes accentuated in this case.

5.6 - SUMMARY AND CONCLUSIONS

Eight field deposition tests were performed to study hydraulic fills. The diameter of the spigots and their position around the pipeline were varied in order to obtain different values of flow rate and slurry concentration, respectively. Rotation of the spigots around the pipeline section has proved to be a practical and economical way of varying the slurry concentration, however it also changed the slurry composition. The use of spigots coming out of the top of the pipeline was particularly advantageous for cell construction, as carried out at Syncrude Canada Ltd., but aggravated problems of dyke erosion by having the slurry jet more exposed to the wind.

The monitoring of the tests was adequate, especially considering the harsh weather conditions during most of the tests. It could be improved by measuring the flow velocity or flow rate at the spigots more accurately and by sampling the slurry more often and in a larger container.

The detailed observation of the flow conditions on the beach permitted a better understanding of the variability of the beach parameters. A phenomenon of deep channelling on the beach with all the sediment being transported to the pond was observed and described. Although references to this phenomenon in the literature are not known, it is common at that particular site. The causes and characteristics of such channels need to better studied as they can have an important impact on the operation of decant facilities and on the lifetime of the impoundment.

The results of the field tests showed an increase in beach slope for higher slurry concentrations and for larger mean grain sizes. However, the effect of flow rate on the slope could not be assessed. The field tests results were also not conclusive in relation to the effects of the various factors on the beach density, because a complete set of data was available for only three tests and these three tests included many variables as they had different values of concentration, grain size and distance from the spigots to the final surface of the beach. Beach density became smaller with distance from the discharge point. Particle size sorting was also observed along the beach, with mean grain size decreasing towards the downstream. The coefficient of uniformity C_U and the amount of fines increased with distance from the spigots. It is interesting to observe that the exact opposite trend in grain size distribution was observed within the deep channels of Test 2.

The field results showed that fines capture efficiency was improved for higher slurry concentrations and was minimum inside the deep channels. After a certain value of slurry concentration (around 60%), a significant increase in the efficiency of fines capture was observed, which coincides with a decrease in beach density and relative density.

5.7 - REFERENCES

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TABLE 5.1 - Summary of field deposition tests performed at Syncrude's tailings dam

TEST nº	YEAR	DEPOSITION PERIOD		LOCATION	LINE	SPIGOT DIAMETER	SPIGOT POSITION
		START	FINISH				
0	1986	December, 2	December, 9	Cell 11	5	6"	side/bottom
1	1988	August, 22	August, 24	Cell 6	5	6"	bottom
2	1988	September, 15	September, 30	Cell 6	5	6"	centre
3	1988	October, 23	October, 29	Cell 8	5	6"	top
4	1988	November, 25	December, 2	Cell 7	5	6"	top
5	1988	November, 18	November, 20	Cell 30	3	3"	centre
6	1988	November, 24	December, 5	Cell 18	1	3"	centre
7	1988	December, 7	December, 12	Cell 18	1	3"	bottom

TABLE 5.2 - Grain size parameters of beach samples - Test nº 0

Sample	Dist.(m)	w (%)	D10 (mm)	D50 (mm)	D60 (mm)	D90 (mm)	CU	D90/D10	%<#200
D1	52.0	21.3	0.070	0.185	0.203	0.380	2.90	5.43	11.0
D3	107.5	13.9	0.053	0.162	0.182	0.285	3.43	5.38	13.5
D4	72.5	22.6	0.040	0.167	0.185	0.250	4.63	6.25	20.0
D5	72.0	21.5	0.087	0.182	0.200	0.330	2.30	3.79	8.5
D8	19.0	4.6	0.080	0.192	0.210	0.390	2.63	4.88	9.5
D11	63.5	4.7	0.055	0.143	0.164	0.240	2.98	4.36	14.7
D12	99.0	23.8	0.052	0.138	0.160	0.235	3.08	4.52	15.0
D13	170.0	32.7	0.039	0.132	0.152	0.230	3.90	5.90	16.2
D14A	75.0	23.8	0.079	0.170	0.187	0.250	2.37	3.16	7.0
D14B	75.0	24.4	0.081	0.173	0.190	0.300	2.35	3.70	6.0
D14C	75.0	24.6	0.078	0.164	0.180	0.240	2.31	3.08	7.0
B15	52.0	23.1	0.080	0.163	0.182	0.260	2.28	3.25	6.0
D15B	52.0	20.4	0.083	0.179	0.197	0.325	2.37	3.92	5.0
B16	29.0	21.3	0.078	0.188	0.208	0.410	2.67	5.26	9.5
B17	19.0	11.2	0.081	0.167	0.176	0.287	2.17	3.54	6.0
B18	7.0	8.3	0.074	0.167	0.174	0.250	2.35	3.38	10.0
B19	26.0	18.9	0.080	0.170	0.188	0.325	2.35	4.06	6.0
B20	9.0	5.3	0.076	0.181	0.200	0.340	2.63	4.47	9.5
B21	18.0	10.8	0.042	0.180	0.200	0.340	4.76	8.10	16.0
B22	9.0	7.2	0.054	0.182	0.200	0.410	3.70	7.59	14.5
AVERAGE VALUES			0.068	0.169		0.304	2.91	4.70	10.6
B21 - 2"	18.0	12.3	0.076	0.178	0.194	0.312	2.55	4.11	4.5
B21 - 4"	18.0	11.4	0.075	0.177	0.194	0.322	2.59	4.29	4.5
B21 - 6"	18.0	13.1	0.078	0.185	0.206	0.400	2.64	5.13	4
B21 - 8"	18.0	15.6	0.082	0.167	0.184	0.250	2.24	3.05	4.5
B21 -10"	18.0	11.4	0.092	0.192	0.213	0.420	2.32	4.57	3.0
B21 -12"	18.0	17.2	0.077	0.160	0.180	0.270	2.34	3.51	8.5
B21 -14"	18.0	16.7	0.081	0.170	0.187	0.248	2.31	3.06	6.0
B21 -16"	18.0	20.7	0.074	0.160	0.180	0.265	2.43	3.58	10.0
B21 -18"	18.0	21.9	0.081	0.163	0.180	0.242	2.22	2.99	6.0
B21 -20"	18.0	17.6	0.092	0.192	0.213	0.400	2.32	4.35	4.0
B21 -22"	18.0	21.0	0.082	0.182	0.210	0.430	2.56	5.24	5.5
B21 -24"	18.0	18.7	0.084	0.186	0.210	0.430	2.50	5.12	5.5

TABLE 5.3 - Density and moisture content of beach samples - Test 0

SAMPLE #	Distance (m)	Moist. Cont. (%)	Wet Dens. (g/cm3)	Dry Dens. (g/cm3)
T0 SG1	52.0	21	1.79	1.479
T0 SG2	80.0	21	1.86	1.537
T0 SG3	107.5	21	1.80	1.488
T0 SG4	72.5	21	1.91	1.579
T0 SG5	72.0	21	1.81	1.496
T0 SG6	118.0	22	1.90	1.557
T0 SG6b	118.0	14		
T0 SG8	19.0	15		
T0 SG11	63.5	20	1.91	1.592
T0 SG12	99.0	20	1.82	1.517
T0 SG13	170.0	24	2.07	1.669
T0 SG14	75.0	21	1.92	1.587
T0 SG16	29.0	21		
T0 SG18	7.0	14		
T0 SG20	9.0	14		
T0 SG21	18.0	16		
AVERAGE VALUES		19		1.543

TABLE 5.4 - Input parameters - Field Test nº 1

Location	Date	Time	Vel. (m/s)	Cw (%)	%bitumen	%< 75 μ	%< 22 μ
Feed	Aug. 22, 88	18:05 hs		55.5	0.35	21.9	9.3
		20:00 hs		55.1	0.53	24.0	7.1
	Aug. 23, 88	08:30 hs		46.7	0.25	18.9	8.6
		10:30 hs		43.4	0.19	14.2	7.2
		12:30 hs		44.7	0.26	23.7	12.1
		15:30 hs		43.9	0.22	18.4	8.7
		16:30 hs		36.4	0.23	15.9	8.6
		18:30 hs		41.3	0.30	26.0	16.3
	Aug. 24, 88	08:30 hs		36.4	0.30	18.2	9.9
		10:30 hs		42.5	0.27	20.5	10.2
		13:00 hs		45.7	0.27	22.9	12.6
		15:00 hs		49.2	0.23	16.9	8.2
		17:00 hs		49.8	0.27	17.6	7.4
AVERAGE VALUES				45.4 ± 6.0	0.28 ± 0.08	19.9 ± 3.5	9.7 ± 2.6
Spigot 1	Aug. 22, 88	17:30 hs		72.3	0.22	12.0	3.9
		19:30 hs		66.9	0.18	14.0	4.4
	Aug. 23, 88	08:30 hs		69.7	0.16	7.8	2.9
		10:30 hs		68.7	0.11	7.0	0.6
		12:30 hs		65.8	0.17	12.2	5.3
		14:30 hs		63.9	0.16		
		16:30 hs		61.5	0.22	11.6	5.4
		18:30 hs	3.6	67.6	0.20	10.5	2.9
	Aug. 24, 88	09:00 hs		70.9	0.13	7.0	3.0
		10:30 hs		69.5	0.19	10.7	2.4
		13:00 hs	3.0	69.7	0.22	10.2	1.8
		15:00 hs	3.1	70.6	0.17	8.3	3.2
		17:00 hs	2.2	72.6	0.17	6.9	2.5
AVERAGE VALUES				68.4 ± 3.2	0.18 ± 0.03	9.9 ± 2.4	3.2 ± 1.4
Spigot 2	Aug. 22, 88	17:45 hs		64.0	0.24	17.3	7.5
		19:30 hs		64.9	0.18	16.8	5.2
	Aug. 23, 88	08:30 hs		66.8	0.13	5.8	0.0
		14:30 hs	3.0	60.3	0.13		
		18:30 hs		62.3	0.19	12.9	5.3
	Aug. 24, 88	09:00 hs		68.1	0.14	8.7	2.2
		10:30 hs		65.2	0.19	8.2	1.7
		15:00 hs		66.6	0.14	9.2	3.3
		17:00 hs		74.4	0.15	8.8	3.5
AVERAGE VALUES				65.8 ± 4.0	0.17 ± 0.04	11.0 ± 4.2	3.6 ± 2.4

TABLE 5.4 (Cont.)

Location	Date	Time	Vel. (m/s)	Cw (%)	%bitumen	%< 75 µ	%< 22 µ
Spigot 3	Aug. 22, 88	17:50 hs		65.4	0.26	16.6	7.8
		19:30 hs		70.7	0.14	12.9	5.1
	Aug. 23, 88	08:30 hs		65.2	0.13	8.3	2.5
		14:30 hs	3.3	66.1	0.14	9.9	3.6
		18:30 hs	4.1	65.8	0.15	9.7	2.6
	Aug. 24, 88	09:00 hs		67.2	0.15	7.4	1.9
		10:30 hs		65.6	0.20	10.8	2.4
		15:00 hs		68.6	0.14	8.7	3.1
		17:00 hs		68.8	0.18	6.2	1.6
AVERAGE VALUES				67.0±1.9	0.17±0.04	10.1 ±3.1	3.4±2.0
Spigot 4	Aug. 22, 88	17:55 hs		64.6	0.27	18.2	8.7
		19:30 hs		67.0	0.18	14.9	6.3
	Aug. 23, 88	08:30 hs		61.9	0.13	10.5	4.5
		14:30 hs		65.2	0.13	9.4	2.8
		18:30 hs		61.3	0.15	13.9	5.9
	Aug. 24, 88	09:00 hs		45.5	0.19	5.9	0.0
		10:30 hs		56.1	0.22	13.9	4.9
		15:00 hs		63.8	0.16	7.2	0.6
		17:00 hs		62.0	0.21	10.7	5.3
AVERAGE VALUES				60.8±6.5	0.18±0.05	11.6±3.9	4.3±3.0
Spigot 5	Aug. 22, 88	18:00 hs		66.7	0.19	14.8	5.8
		19:30 hs		67.1	0.15	14.1	5.2
	Aug. 23, 88	08:30 hs		57.1	0.13	11.7	4.6
		12:30 hs		61.3	0.15	14.0	6.1
		16:30 hs		35.2	0.29	22.0	11.6
		18:30 hs	4.2	63.7	0.12	11.4	4.1
		19:00 hs		62.5	0.14	11.4	4.9
	Aug. 24, 88	09:00 hs		64.7	0.14	10.7	2.0
		10:30 hs		42.3	0.31	20.4	8.5
		13:00 hs		63.9	0.20	14.0	5.2
		15:00 hs		64.9	0.15	5.6	0.0
AVERAGE VALUES				59.0±10.5	0.18±0.06	13.7±4.5	5.3±3.0
OVERALL AVERAGES				**64.2%**	**0.18%**	**11.2%**	**4.0%**

Table 5.5 - Grain Size parameters of undisturbed samples - Field Test nº 1

SAMPLE	Dist.(m)	D10 (mm)	D50 (mm)	D60 (mm)	D90 (mm)	CU	%<#200
T1 SG 1-1	6	0.111	0.183	0.196	0.256	1.77	5.1
T1 SG 1-2*	6	0.150	0.203	0.224		1.49	1.6
T1 SG 1-3*	6	0.127	0.191	0.203	0.303	1.60	3.4
T1 SG 2	25	0.124	0.192	0.205	0.339	1.65	4.3
T1 SG 3-1	49	0.131	0.213	0.236	0.598	1.80	2.5
T1 SG 3-2*	49	0.102	0.185	0.200	0.359	1.96	5.1
T1 SG 4-1	77	0.097	0.180	0.196	0.335	2.02	6.2
T1 SG 4-2*	77	0.083	0.166	0.183	0.278	2.20	7.9
T1 SG 4-3*	77	0.078	0.157	0.168	0.224	2.15	8.8
T1 SG 5*	102	0.106	0.189	0.206	0.469	1.94	5.0
T1 SG 6*	124	0.065	0.163	0.177	0.288	2.72	11.1
T1 SG 7*	153	0.086	0.168	0.183	0.339	2.13	6.3
T1 SG 10	7	0.127	0.189	0.202	0.288	1.59	3.5
T1 SG 11	34	0.129	0.193	0.205	0.298	1.59	3.4
T1 SG 12	63	0.102	0.183	0.199	0.324	1.95	5.6
T1 SG 13-1	99	0.116	0.193	0.208	0.359	1.79	3.1
T1 SG 15	100	0.083	0.174	0.189	0.308	2.28	7.9
T1 SG 16	14	0.127	0.186	0.199	0.265	1.57	3.3
T1 SG 17-1	7	0.135	0.193	0.206	0.319	1.53	2.9
T1 SG 17-2*	7	0.124	0.189	0.201	0.300	1.62	3.4
T1 SG 18	34	0.135	0.196	0.209	0.346	1.55	3.2
T1 SG 19-1	67	0.061	0.171	0.189	0.265	3.10	11.6
T1 SG 19-2*	67	0.096	0.179	0.193	0.298	2.01	5.5
T1 SG 20-1	99	0.089	0.168	0.186	0.261	2.09	6.5
T1 SG 20-2*	99	0.098	0.175	0.186	0.246	1.90	5.7
T1 SG 23	96	0.097	0.175	0.189	0.250	1.95	4.8
T1 SG 24	97	0.084	0.166	0.184	0.272	2.19	7.3
T1 SG 25	63	0.129	0.195	0.206	0.326	1.60	3.0
T1 SG 26	36	0.127	0.186	0.199	0.255	1.57	2.8
T1 SG 27	10	0.120	0.186	0.197	0.268	1.64	3.5
T1 SG 28	16	0.136	0.193	0.205	0.298	1.51	3.0
T1 SG 29	25	0.090	0.171	0.185	0.252	2.06	6.6
T1 SG 30*	14	0.115	0.201	0.220	0.432	1.91	2.0
T1 SG 31	149	0.091	0.168	0.178	0.239	1.96	5.7
T1 SG 32	150	0.103	0.168	0.177	0.228	1.72	4.8
T1 SG 34	100	0.108	0.176	0.189	0.252	1.75	4.5
T1 SG 35	100	0.090	0.168	0.180	0.242	2.00	6.0
T1 SG 36*	98	0.118	0.186	0.200	0.346	1.69	4.0
AVERAGE VALUES (except *)			0.183 ± 0.012			1.85 ± 0.34	4.8 ± 2.1

TABLE 5.6 - Density and moisture content of beach samples - Test 1

SAMPLE #	Distance (m)	Moist. Content (%)	Wet Density (g/cm3)	Dry Density (g/cm3)
T1 SG 1 1/3	6	4.7	1.487	1.420
T1 SG 1 2/3*	6	5.1	1.604	1.526
T1 SG 1 3/3*	6	6.4	1.569	1.474
T1 SG 2	25	4.6	1.489	1.424
T1 SG 3 1/3	49	12.5	1.779	1.581
T1 SG 3 2/3*	49	15.2	1.715	1.489
T1 SG 4 1/3	77	11.1	1.639	1.475
T1 SG 4 2/3*	77	22.9	1.821	1.482
T1 SG 4 3/3*	77	17.5	1.754	1.492
T1 SG 5*	102	15.7	1.783	1.541
T1 SG 6*	124	19.5	1.848	1.547
T1 SG 7*	153	20.4	1.899	1.577
T1 SG 10*	7	5.8	1.371	1.296
T1 SG 11	34	6.4	1.546	1.454
T1 SG 12	63	12	1.499	1.339
T1 SG 13	99	13.2	1.658	1.464
T1 SG 15	100	17	1.703	1.455
T1 SG 16	14	5.7	1.515	1.434
T1 SG 17 1/2	7	15.9	1.846	1.592
T1 SG 17 2/2*	7	7.6	1.614	1.500
T1 SG 18	34	4.7	1.494	1.427
T1 SG 19 1/2	67	14	1.701	1.492
T1 SG 19 2/2*	67	13.8	1.696	1.491
T1 SG 20 1/2	99	14.1	1.614	1.415
T1 SG 20 2/2*	99	22.2	1.927	1.577
T1 SG 23	96	15.6	1.824	1.578
T1 SG 24	97	14.7	1.721	1.501
T1 SG 25	63	5.9	1.564	1.477
T1 SG 26	36	6.7	1.557	1.460
T1 SG 27	10	5.6	1.443	1.367
T1 SG 28	16	4.5	1.447	1.385
T1 SG 29	25	9.1	1.546	1.418
T1 SG 30*	14	6.2	1.648	1.553
T1 SG 31	149	17.2	1.769	1.509
T1 SG 32	150	7.3	1.537	1.433
T1 SG 34	100	9.3	1.595	1.459
T1 SG 35	100		1.579	
T1 SG 36*	98	9.1	1.625	1.490
AVERAGE VALUES (except *)		9.9 ± 4.5	1.597 ± 0.126	1.452 ± 0.071

TABLE 5.7 - Input parameters - Field Test nº 2

Location	Date	Time	Vel.(m/s)	Cw (%)	%bitum	Microtrac		Hydrometer	
						%< 75 µ	%< 22 µ	%< 75 µ	%< 22 µ
Feed	Sept.15, 88	12:00 hs		47.2		29.9	12.1	27.0	10.8
		15:00 hs		44.9		38.9	16.6	37.1	18.6
		17:00 hs		52.0		38.8	15.0	39.5	16.7
		19:00 hs		56.5		34.6	16.0	29.9	12.1
	Sept.16, 88	10:00 hs		45.5		21.5	8.9		
		12:00 hs		48.2		36.9	16.6		
		14:00 hs		54.1		27.9	12.3		
		16:00 hs		49.1	0.51	36.9	16.3		
	Sept.20, 88	10:00 hs		48.1	0.34	28.3	16.1		
		12:00 hs		52.2	0.29	28.8	17.5		
		14:00 hs		45.7	0.30	26.1	14.5		
		16:00 hs		55.0	0.25	24.8	13.3		
		18:00 hs		54.5	0.25	22.6	10.8		
	Sept.21, 88	10:00 hs		56.8	0.30	25.1	14.1		
		12:00 hs		47.0	0.28	30.9	16.0		
		14:00 hs		59.3	0.27	21.6	12.2		
		16:00 hs		49.3	0.28	29.9	16.5		
		18:00 hs				28.1	16.1		
	Sept.22, 88	10:00 hs		55.9	0.28	24.2	12.1		
		13:00 hs		54.4	0.27	25.9	12.8		
		15:00 hs		59.0	0.29	26.4	12.9		
		17:00 hs		59.5	0.27	25.5	6.0		
	Sept.27, 88	09:00 hs		55.9	0.34	30.2	15.1		
		11:00 hs		48.5	0.35	27.5	11.9		
		13:00 hs		44.1	0.58	32.8	14.0		
		15:00 hs		51.1	0.31	32.9	15.1		
	Sept.28, 88	09:00 hs		48.2	0.36	34.6	15.5		
		11:00 hs		50.7	0.37	36.1	18.4		
		13:00 hs		51.3	0.35	35.8	18.2		
		15:00 hs		56.9	0.37	29.2	16.2		
		18:00 hs		53.9	0.29	25.1	12.5		
	Sept.29, 88	09:00 hs		49.4	0.40	33.8	13.6		
	Sept.30, 88	13:00 hs		59.3	0.27	20.9	10.3		
		15:00 hs		57.5	0.25	19.8	5.9		
		17:00 hs		57.2	1.11	20.1	9.3		
AVERAGE VALUES				52.3±4.7	0.35±0.17	28.9±5.5	13.7±3.1		

TABLE 5.7 (Cont.)

Location	Date	Time	Vel.(m/s)	Cw (%)	% bitum	Microtrac		Hydrometer	
						%< 75 µ	%< 22 µ	%< 75 µ	%< 22 µ
Spigot 1	Sept.15, 88	13:00 hs	3.0	47.0		26.4	12.5		
		15:00 hs	2.0	*		33.3	13.4		
		17:00 hs	3.0	50.1		39.0	18.9		
		19:00 hs		52.5		33.5	13.2		
	Sept.16, 88	10:00 hs	2.5	50.6		32.2	13.4		
		12:00 hs	2.8	46.0		42.9	19.0	38.7	14.5
		14:00 hs	2.7	54.8		27.8	11.2	28.5	12.6
		16:00 hs	2.6	52.3		28.6	9.8	27.9	11.2
	Sept.22, 88	10:00 hs		57.6					
		13:00 hs	3.0	56.8					
		15:00 hs	3.5	61.0					
		17:00 hs	3.5	57.0					
	Sept.27, 88	9:00 hs	3.5	58.2					
		11:00 hs	3.0	47.7					
		13:00 hs	3.0	51.4					
		15:00 hs	3.7	51.9					
	Sept.28, 88	9:00 hs	2.2	45.3					
		11:00 hs	2.2	54.6					
		13:00 hs	2.2	59.6					
		15:00 hs	2.7	58.6					
		18:00 hs	2.5	51.5					
	Sept.29, 88	9:00 hs		49.3					
	Sept.30, 88	13:00 hs		63.9					
		15:00 hs		58.3					
AVERAGE VALUES				53.7±5.1		33.0±5.7	13.9±3.3		
Spigot 2	Sept.15, 88	13:00 hs	3.5	47.0		28.3	13.4		
		15:00 hs		46.8		30.0	9.7		
		17:00 hs		53.7		33.4	14.2		
		19:00 hs		57.5		31.6	12.1		
	Sept.16, 88	10:00 hs		*		35.6	14.9		
		12:00 hs		42.0		44.4	20.4		
		14:00 hs		59.3		23.3	9.2	20.4	9.4
		16:00 hs		*		28.2	11.6		
	Sept.22, 88	10:00 hs		57.9					
		13:00 hs		59.3					
		15:00 hs		61.7					
		17:00 hs		59.4					
	Sept.27, 88	9:00 hs		57.2					
		11:00 hs		51.6					
		13:00 hs		54.3					
		15:00 hs		54.4					
	Sept.28, 88	9:00 hs		51.1					
		11:00 hs		54.7					
		13:00 hs		56.6					
		15:00 hs		59.9					
		18:00 hs		52.5					

TABLE 5.7 (Cont.)

Location	Date	Time	Vel.(m/s)	Cw (%)	%bitum	Microtrac		Hydrometer	
						%< 75 µ	%< 22 µ	%< 75 µ	%< 22 µ
Spigot 2	Sept.29, 88	9:00 hs		52.2					
	Sept.30, 88	13:00 hs		62.4					
		15:00 hs		59.2					
		17:00 hs		62.1					
AVERAGE VALUES				55.3±5.3		31.9±6.3	13.2±3.5		
Spigot 3	Sept.15, 88	13:00 hs	3.4	48.3		28.7	11.8		
		15:00 hs		47.7		33.6	12.2		
		17:00 hs		51.8		35.2	14.5		
		19:00 hs		55.0		31.4	12.0		
	Sept.16, 88	10:00 hs		46.1		39.5	18.7	33.1	14.2
		12:00 hs		38.2		55.1	23.5	37.4	17.8
		14:00 hs		*		25.9	10.1		
		16:00 hs		*		29.5	12.5		
	Sept.22, 88	10:00 hs		57.1					
		13:00 hs		58.2					
		15:00 hs		60.7					
		17:00 hs		55.8					
	Sept.27, 88	9:00 hs		58.8					
		11:00 hs		48.5					
		13:00 hs		54.0					
		15:00 hs		57.0					
	Sept.28, 88	9:00 hs		58.0					
		13:00 hs		59.0					
		15:00 hs		64.3					
		18:00 hs		54.3					
	Sept.29, 88	9:00 hs		54.2					
	Sept.30, 88	13:00 hs		63.9					
		15:00 hs		59.6					
		17:00 hs		62.0					
AVERAGE VALUES				55.1±6.3		34.9±9.2	14.4±4.5		
Spigot 4	Sept.15, 88	13:00 hs	3.3	45.3		30.0	13.3		
		15:00 hs		53.8		29.3	10.4		
		17:00 hs		51.7		34.2	14.4		
		19:00 hs		55.9		31.1	13.8		
	Sept.16, 88	10:00 hs		38.2		37.9	18.3	40.2	14.9
		12:00 hs		40.0		39.4	15.3	34.6	17.1
		14:00 hs		*		25.3	9.4		
	Sept.22, 88	10:00 hs		62.0					
		13:00 hs		63.2					
		15:00 hs		64.4					
		17:00 hs		60.7					
	Sept.27, 88	9:00 hs		58.1					
		11:00 hs		50.5					
		13:00 hs		53.5					
		15:00 hs		53.3					

TABLE 5.7 (Cont.)

Location	Date	Time	Vel.(m/s)	Cw (%)	% bitum	Microtrac		Hydrometer	
						%< 75 µ	%< 22 µ	%< 75 µ	%< 22 µ
Spigot 4	Sept.28, 88	9:00 hs		50.8					
		11:00 hs		55.9					
		13:00 hs		54.8					
		15:00 hs		61.3					
		18:00 hs		49.6					
	Sept.29, 88	9:00 hs		53.8					
	Sept.30, 88	13:00 hs		66.1					
		15:00 hs		62.9					
		17:00 hs		67.2					
AVERAGE VALUES				47.5					
Spigot 5	Sept.15, 88	13:00 hs	3.0	55.7		24.0	8.4		
		15:00 hs		52.4		29.8	9.4		
		17:00 hs		55.8		36.6	19.0		
		19:00 hs		56.7		30.1	12.2		
	Sept.16, 88	10:00 hs		*		27.4	11.7		
		12:00 hs		43.5		37.6	15.4	36.7	16.5
		14:00 hs		*		23.4	9.7	27.4	9.4
	Sept.22, 88	10:00 hs		65.2					
		13:00 hs		64.3					
		15:00 hs		66.7					
		17:00 hs		63.9					
	Sept.27, 88	9:00 hs		61.5					
		11:00 hs		55.6					
		13:00 hs		56.5					
		15:00 hs		59.1					
	Sept.28, 88	9:00 hs		55.1					
		11:00 hs		55.7					
		13:00 hs		59.7					
		15:00 hs		66.4					
		18:00 hs		57.0					
	Sept.29, 88	9:00 hs		55.1					
	Sept.30, 88	13:00 hs		68.0					
		15:00 hs		62.5					
		17:00 hs		66.6					
AVERAGE VALUES				59.2±5.9		29.8±5.6	12.3±3.8		
OVERALL AVERAGES				**55.7**		**32.4**			

TABLE 5.8 - Grain size parameters of undisturbed samples - Field Test nº 2

SAMPLE	Dist.(m)	D10 (mm)	D50 (mm)	D60 (mm)	D90 (mm)	CU=D60/D10	%<#200
T2 SG 1*	10.0	0.078	0.158	0.166	0.208	2.13	8.8
T2 SG 2*	18.9	0.064	0.153	0.163	0.209	2.55	11.1
T2 SG 3*	10.0	0.062	0.151	0.161	0.206	2.60	11.6
T2 SG 4*	23.6	0.109	0.186	0.203	0.365	1.86	4.6
T2 SG 5-1*	30.0	0.091	0.155	0.167	0.235	1.84	13.9
T2 SG 5-2*	30.0	0.046	0.135	0.152	0.187	3.30	15.4
T2 SG 6	78.4	0.116	0.168	0.175	0.228	1.51	3.4
T2 SG 7	110.4	0.130	0.193	0.209	0.414	1.61	2.6
T2 SG 8*	12.0	0.063	0.147	0.160	0.227	2.54	11.3
T2 SG 9	154.0	0.083	0.174	0.196	0.384	2.36	7.9
TS SG 10	129.0	0.076	0.157	0.168	0.223	2.21	9.7
T2 SG 11	92.0	0.054	0.137	0.152	0.229	2.81	15.0
T2 SG 12	53.0	0.099	0.180	0.200	0.347	2.02	5.0
T2 SG 13*	11.0	0.090	0.171	0.190	0.341	2.11	6.7
T2 SG 15*	30.1	0.107	0.166	0.180	0.245	1.68	3.4
T2 SG 16-1*	48.2	0.111	0.183	0.200	0.366	1.80	3.0
T2 SG 16-2*	48.2	0.109	0.167	0.180	0.248	1.65	3.5
T2 SG 17	105.0	0.080	0.160	0.180	0.299	2.25	8.1
T2 SG 18	66.0	0.084	0.172	0.189	0.319	2.25	7.2
T2 SG 19	64.0	0.093	0.174	0.190	0.301	2.04	6.2
T2 SG 20	62.0	0.090	0.169	0.187	0.330	2.08	6.4
T2 SG 21	10.0	0.095	0.196	0.221	0.600	2.33	5.9
T2 SG 22	10.0	0.085	0.165	0.184	0.320	2.16	7.3
T2 SG 23*	41.1	0.111	0.177	0.193	0.325	1.74	3.0
AVERAGE VALUES (except *)			0.170 ± 0.016			2.14 ± 0.34	7.1 ± 3.2

TABLE 5.9 - Density and moisture content of undisturbed samples - Test nº 2

Sample #	Distance (m)	Moist.Cont. (%)	Wet Density (g/cm3)	Dry Density (g/cm3)
T2 SG 1 1*	10.0	19.1	1.791	1.504
T2 SG 1 2*		16.5	1.718	1.475
T2 SG 1 3*		23.7	1.804	1.459
T2 SG 2*	18.9	18.1	1.728	1.462
T2 SG 3*	10.0	18.8	1.789	1.505
T2 SG 4*	23.6	7.2	1.659	1.547
T2 SG 5 -1 1*	30.0	11.7	1.726	1.546
T2 SG 5 -1 2*	30.0	21.6	1.853	1.524
T2 SG 5 -1 3*		23.0	1.807	1.469
T2 SG 5 -2 1*		25.4	1.817	1.449
T2 SG 5 -2 2*		25.0	1.846	1.477
T2 SG 5 -2 3*		24.4	1.884	1.514
T2 SG 5 -2 4*		24.5	1.910	1.534
T2 SG 6	78.4	6.6	1.477	1.385
T2 SG 7	110.4	13.0	1.717	1.520
T2 SG 8*	12.0	9.6	1.626	1.483
T2 SG 9 -1	154.0	10.4	1.617	1.465
T2 SG 9 -2		11.1	1.671	1.504
T2 SG 9 -3		17.7	1.916	1.627
T2 SG 9 -4		17.1	1.844	1.575
T2 SG 10 -1	129.0	11.1	1.591	1.431
T2 SG 10 -2		12.5	1.612	1.427
T2 SG 10 -3		17.5	1.767	1.504
T2 SG 11 -1	92.0	12.2	1.692	1.508
T2 SG 11 -2		16.7	1.857	1.592
T2 SG 11 -3		17.5	1.803	1.534
T2 SG 11 -4		21.8	1.784	1.464
T2 SG 12	53.0	7.5	1.638	1.524
T2 SG 13*	11.0	8.9	1.664	1.528
T2 SG 14		7.6	1.575	1.463
T2 SG 15*	30.1	5.7	1.526	1.444
T2 SG 16 1/2*	48.2	19.0	1.838	1.544
T2 SG 16 2/2*	48.2	19.3	1.812	1.519
T2 SG 17	105.0	9.3	1.625	1.486
T2 SG 18	66.0	8.9	1.620	1.487
T2 SG 19	64.0	11.6	1.695	1.519
T2 SG 20	62.0	8.1	1.661	1.536
T2 SG 21	10.0	7.0	1.496	1.398
T2 SG 22 -1	10.0	10.3	1.572	1.426
T2 SG 22 -2		9.8	1.709	1.556
T2 SG 22 -3		15.0	1.806	1.571
T2 SG 22 -4		15.3	1.751	1.519
T2 SG 23*	41.1	7.0	1.592	1.487
AVERAGE (except*)	OUT-OF-CHANNEL	11.0 ± 3.3	1.665 ± 0.081	1.499 ± 0.045
	IN-CHANNEL	9.1 ± 3.4	1.590 ± 0.121	1.456 ± 0.068
	OVERALL	10.5 ± 3.3	1.647 ± 0.092	1.489 ± 0.051

TABLE 5.10 - Input parameters - Field Test nº 3

Location	Date	Time	Cw (%)	% bitumen	%< 75 μ	%< 22 μ
Feed	Oct. 12, 88	19:00 hs	55.06	0.26	26.4	12.9
	Oct. 17, 88	15:30 hs	54.65	0.23	20.2	10.3
		18:00 hs	62.27	0.34	22.6	11.2
	Oct. 18, 88	10:30 hs	44.53	0.24	26.6	12.6
AVERAGE VALUES			54.1	0.27	24.0	11.8
Spigot 1	Oct. 12, 88	17:00 hs	34.41	0.37	52.4	31.6
	Oct. 17, 88	15:00 hs	37.38	0.53	47.5	26.9
		18:00 hs	48.89	0.41	33.0	16.7
	Oct. 18, 88	10:30 hs	24.82	0.42	55.9	28.8
AVERAGE VALUES			36.4	0.43	47.2	26.0
Spigot 2	Oct. 12, 88	17:15 hs	44.71	0.28	38.2	18.2
	Oct. 17, 88	15:00 hs	41.29	0.5	46.3	27.9
		18:00 hs	51.17	0.41	30.3	10.8
	Oct. 18, 88	10:30 hs	29.6	0.35	42.4	20.1
AVERAGE VALUES			41.7	0.39	39.3	19.3
Spigot 3	Oct. 12, 88	17:30 hs	46.23	0.29	36.5	19.2
	Oct. 17, 88	15:00 hs	38.5	0.46	39.4	21.4
		18:00 hs	56.32	0.39	28.6	14.3
	Oct. 18, 88	10:30 hs	31.23	0.32	39.4	17.3
AVERAGE VALUES			43.1	0.37	36.0	18.1
Spigot 4	Oct. 12, 88	17:35 hs	42.25	0.29	38.7	19.1
	Oct. 17, 88	15:00 hs	46.43	0.42	33.3	16.0
		18:00 hs	54.56	0.34	25.2	8.8
	Oct. 18, 88	10:30 hs	32.89	0.29	41.9	18.4
AVERAGE VALUES			44.0	0.34	34.8	15.6
Spigot 5	Oct. 12, 88	18:00 hs	38.32	0.31	47.3	26.3
	Oct. 17, 88	15:00 hs	37.38	0.53	32.4	17.4
		15:00 hs	45.44	0.43	24.9	
		18:00 hs	55.74	0.38		11.5
AVERAGE VALUES			44.2	0.41	34.9	18.4
OVERALL AVERAGES			**41.9%**	**0.39%**	**38.4%**	**19.5%**

TABLE 5.11 - Input parameters - Field Test nº 4

Location	Date	Time	Cw (%)	%bitumen	%< 75 μ	%< 22 μ
Spigot 1	Nov. 28, 88	11:30 hs	34.3	3.03	48.6	25.4
		13:30 hs	26.7	0.43	59.4	31.3
		16:30 hs	38.1	0.42	50.2	28.3
	Nov. 29, 88	14:00 hs	32.2	-	59.4	32.3
		16:30 hs	30.1	0.39	57.6	29.8
	Nov. 30, 88	11:30 hs	33.3	0.53	51.4	27.4
		13:30 hs	28.6	0.69	52.2	28.2
		15:30 hs	37.6	0.82	47.8	21.4
	Dec. 01, 88	16:00 hs	24.2	0.40		
	Dec. 02, 88	9:00 hs	37.1	0.36		
		13:30 hs	50.9	0.50		
		15:00 hs	44.2	0.38	40.2	19.4
AVERAGE VALUES			34.8±7.5	0.72±0.78	51.9±6.3	27.1±4.3
Spigot 2	Nov. 28, 88	11:30 hs	35.7	0.37	48.0	22.2
		13:30 hs	30.9	0.36	45.9	21.7
		16:30 hs	43.7	0.40	43.2	21.3
	Nov. 29, 88	14:00 hs	-	0.61	56.3	30.8
		16:30 hs	25.9	0.36	53.0	25.8
	Nov. 30, 88	11:30 hs	41.8	0.43	40.2	19.2
		13:30 hs	33.0	0.57	49.0	24.3
		15:30 hs	39.9	0.67	44.1	20.8
	Dec. 01, 88	16:00 hs	29.9	0.38		
	Dec. 02, 88	9:00 hs	44.6	0.34		
		13:30 hs	-	0.74		
		15:00 hs	48.0	-	34.5	15.1
AVERAGE VALUES			37.3±7.3	0.48±0.14	46.0±6.6	22.4±4.4
Spigot 3	Nov. 28, 88	11:30 hs	38.7	0.39	49.2	25.1
		13:30 hs	35.1	0.32	41.8	20.3
		16:30 hs	42.6	-	43.2	21.5
	Nov. 29, 88	14:00 hs	34.3	0.56	57.5	31.6
		16:30 hs	34.0	0.35	47.1	21.1
	Nov. 30, 88	11:30 hs	45.1	0.41	34.8	15.5
		13:30 hs	35.5	0.51	44.1	21.4
		15:00 hs	43.2	0.57	30.2	8.8
AVERAGE VALUES			38.6±4.5	0.44±0.10	43.5±8.4	20.7±6.6
OVERALL AVERAGES			**36.9%**		**47.1%**	

TABLE 5.12 - Grain size parameters of undisturbed samples - Field Test nº 4

SAMPLE	Dist.(m)	D10 (mm)	D50 (mm)	D60 (mm)	D90 (mm)	CU=D60/D10	%<#200
T4 SG 1	23	0.074	0.154	0.174	0.240	2.35	10.2
T4 SG 2	67	0.076	0.155	0.175	0.241	2.30	10.0
T4 SG 3	110	0.055	0.142	0.161	0.236	2.93	13.6
T4 SG 4	151	0.069	0.133	0.144	0.206	2.09	10.8
T4 SG 5	196	0.069	0.139	0.153	0.223	2.22	11.0
T4 SG 7	25	0.078	0.150	0.167	0.228	2.14	8.9
T4 SG 8	61	0.060	0.128	0.138	0.200	2.30	12.8
T4 SG 9	108	0.074	0.118	0.128	0.173	1.73	10.3
T4 SG 10	147	0.048	0.114	0.124	0.178	2.58	19.3
T4 SG 11	188	0.044	0.118	0.128	0.176	2.91	18.2
T4 SG 12	110	0.066	0.150	0.171	0.243	2.59	11.3
T4 SG 14	23	0.067	0.152	0.171	0.232	2.55	11.0
T4 SG 14a*	23	0.073	0.145	0.162	0.228	2.22	10.5
T4 SG 16	102	0.056	0.132	0.142	0.200	2.54	13.3
T4 SG 22*	31	0.085	0.168	0.186	0.262	2.19	7.2
T4 SG 23*	72	0.087	0.142	0.154	0.208	1.77	4.9
T4 SG 24*	103	0.061	0.116	0.126	0.176	2.07	13.8
T4 SG 25*	154	0.060	0.142	0.160	0.233	2.67	12.3
T4 SG 26*	196	0.064	0.147	0.160	0.218	2.50	11.7
AVERAGE VALUES (except*)			0.137±0.015			2.40 ± 0.33	12.4 ± 3.1

TABLE 5.13 - Density and moisture content of undisturbed samples - Test nº 4

Sample #	Distance (m)	Moist.Cont. (%)	Wet Density (g/cm3)	Dry Density (g/cm3)
T4 SG 1	23	15.3	1.810	1.570
T4 SG 2	67	16.4	1.667	1.432
T4 SG 3	110	27.9	1.822	1.425
T4 SG 4	151	29.7	1.795	1.384
T4 SG 5	196	27.3	1.847	1.450
T4 SG 7	25	10.0	1.675	1.523
T4 SG 8	61	22.7	1.919	1.564
T4 SG9	108	26.4	1.810	1.432
T4 SG 10	147	27.9	1.840	1.438
T4 SG 11	188	27.4	1.851	1.453
T4 SG 12	110	21.4	1.862	1.534
T4 SG 14	23	15.0	1.747	1.519
T4 SG 14 a*	23	13.6	1.576	1.387
T4 SG 16	102	27.1	1.868	1.470
T4 SG 22*	31	12.8	1.713	1.519
T4 SG 23*	72	29.0	1.812	1.405
T4 SG 24*	103	31.1	1.793	1.368
T4 SG 25*	154	30.8	1.806	1.380
T4 SG 26*	196	24.5	1.898	1.525
AVERAGE (except *)		22.7 ± 6.4	1.809 ± 0.074	1.476 ± 0.059

TABLE 5.14 - Input parameters - Field Test nº 6

Location	Date	Time	Cw (%)	%bitumen	%< 75 μ	%< 22 μ
Spigot 1	Nov. 25, 88	09:30 hs	54.2	0.33		
		13:30 hs	53.3	0.29		
	Nov. 28, 88	11:00 hs	52.7	0.29	29.5	9.9
		14:00 hs	49.9	0.25	28.8	13.2
		17:00 hs	51.6	0.3	34.7	13.3
	Nov. 30, 88	17:00 hs	58.8	1.05	28.5	6.6
	Dec. 01, 88	16:00 hs			20.7	6.0
	Dec. 02, 88	11:00 hs	64.9	0.21		
		13:00 hs	62.4	0.29		
		15:00 hs	64.2	0.25		
		16:00 hs	39.5	0.24		
	Dec. 05, 88	11:30 hs	62.6	0.29	22.2	10.9
AVERAGE VALUES			55.8±7.7	0.34±0.24	27.4±5.2	10.0±3.2
Spigot 4	Nov. 25, 88	09:30 hs	52.5	0.35		
		13:30 hs	52.9	0.31		
	Nov. 28, 88	11:00 hs	52.9	0.25	31.7	15.5
		14:00 hs	46.1	0.27	29.6	11.6
		17:00 hs	51.6	0.34	35.5	15.1
	Nov. 30, 88	17:00 hs	55.9	0.59	32.3	11.5
	Dec. 02, 88	11:00 hs	64.2	0.23		
		13:00 hs	61.0	0.29		
		15:00 hs	63.4	0.21		
		16:00 hs	59.5	0.23		
	Dec. 05, 88	11:30 hs	61.2	0.29	21.5	9.6
AVERAGE VALUES			56.5±5.8	0.31±0.10	30.1±5.3	12.7±2.5

TABLE 5.15 - Input parameters - Field Test nº 7

Location	Date	Time	Cw (%)	%bitumen	%< 75 μ	%< 22 μ
Spigot 3	Dec. 08, 88	14:00 hs	66.45	0.28	16.7	8.6
		15:00 hs	65.75	0.33	12.1	2.4
		16:00 hs	68.14	0.36	12.1	5.4
		17:00 hs	65.26	0.4	16.4	7.6
AVERAGE VALUES			66.4	0.34	14.3	6.0
Spigot 4	Dec. 08, 88	14:00 hs	66.58	0.3	18.6	7.8
		15:00 hs	63.54	0.36	11.2	4.6
		16:00 hs	69.04	0.34	14.3	7.1
		17:00 hs	67.49	0.31	15.4	7.5
	Dec. 12, 88	14:00 hs			14.6	7.5
		15:00 hs	67.65	0.21	13.3	6.3
AVERAGE VALUES			66.9	0.30	14.6	6.8
OVERALL AVERAGES			66.7	0.32	14.5	6.4

TABLE 5.16 - Summary of data from Field Test 0

Location	Hf (m)	L (m)	i av (%)	Samples #	Density (g/cm3)	Void Ratio	D50 (um)	CU	%Fb
prof.I(sp1)	0	95	5.28	S1, S2	1.508	0.757	0.185	2.90	11.0
prof.II(sp2)	0	120	4.33	S3,S4,S8	1.515	0.749	0.174	3.56	14.3
prof.III(sp3)	0	140	4.74	S5, S6	1.509	0.756	0.182	3.00	11.5
prof.IV(sp4)	0	140	4.55				0.181	2.63	9.5
prof.V(sp5)	0	180+	3.71	S11 TO S13	1.593	0.664	0.149	2.92	11.6
prof.3(sp5+6)	0.5	180+	3.76	S11 TO S16	1.591	0.666	0.156	2.86	12.0
prof.VI(sp6)	0.5	100	5.63	S14, S18	1.587	0.670	0.167	2.26	8.0
Average		≈150	4.70	all samples	1.543	0.710	0.169	2.91	10.6

TABLE 5.17 - Summary of data from Field Test nº 1

Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
prof.I(sp1)				68.4	61.7	9.9	1.51	0.49	4.8	0	158	7.28	1 to 8	1.475	0.797		0.192	1.8	4.5	45.5
prof.II(sp2)				65.8	58.6	11.0	1.45	0.55	4.5	0	165	7.33	10 to 14	1.419	0.868		0.190	1.7	3.9	35.5
prof.III(sp3)				67.0	60.3	10.1	1.48	0.51	4.0	0	200	6.40	17 to 21	1.469	0.804		0.182	2.1	6.1	60.4
prof.IV(sp4)				60.8	53.7	11.6	1.34	0.58	3.0	0.5	200+	5.60	22,23,25 to 27	1.471	0.801		0.185	1.7	3.5	30.2
prof.V									3.0	3*	176*	5.40*	31,33							
prof.VI									2.5	2.5*	186*	5.38*	30,32,34							
prof.VII(sp5)				59.0	50.9	13.7	1.30	0.69	2.5	2.5	227	4.85	29,30,32 to 36	1.437	0.844		0.171	1.9	5.5	40.2
prof.VIII									2.5	2.5*	188*	5.37*	36							
Average	45.4	36.4	19.9	64.2	57.0	11.2	1.41	0.56	~3.5	~0/2.5	~200	6.30	all samples:	1.452	0.825		0.183	1.9	4.8	42.9

TABLE 5.18 - Summary of data from Field Test nº 2

Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
prof.I(sp1+2)				54.5	36.8	32.4	1.04	1.12	5	6.5*	110	2.55	1 to 7 (channel)	1.453	0.824		0.181	1.6	3.0	9.3
prof.II(sp2+3)				55.2	36.8	33.4	1.06	1.16	4.2	3.8	170	3.65	9 to 13 (plateau)	1.512	0.753		0.162	2.4	9.4	28.1
prof.III(sp4)				55.3	37.3	32.5	1.06	1.12	5.5	2.8	143	3.92	none (plateau)							
prof.IV(sp5)				59.2	41.6	29.8	1.13	1.03	4.8	1.8	108	4.44	17 to 22 (plateau	1.491	0.777		0.173	2.2	6.9	23.0
Average	52.3	37.2	28.9	55.7	37.7	32.4	1.07	1.12	~5	2.8	~130	4.00	all samples:	1.499	0.768		0.170	2.1	7.1	21.9

TABLE 5.19 - Summary of data from Field Test nº 3

Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
spigot 1				36.4	19.2	47.2	0.67	1.97												
spigot 2				41.7	25.3	39.3	0.77	1.64												
spigot 3				43.1	27.6	36.0	0.80	1.50												
spigot 4				44.0	28.7	34.8	0.81	1.45												
spigot 5				44.2	28.8	34.9	0.82	1.45												
Average	54.1	41.1	24.0	41.9	25.8	38.4	0.77	1.60												

TABLE 5.20 - Summary of data from Field Test nº 4

Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
prof.I(sp1)				34.8	16.7	51.9	0.66	1.79	8.3	7.6	110	2.42	1 to 5	1.452	0.825		0.145	2.4	11.1	21.4
prof.II(sp2)				37.4	20.2	46.0	0.71	1.59	9.5	8.0	170	2.33	7 to 11	1.482	0.788		0.126	2.3	13.9	30.2
prof.III(sp3)				38.5	21.8	43.5	0.73	1.50	10.3	8.9	143	2.04	14 to 18	1.459	0.816		0.142	2.6	12.2	28.0
prof.V(sp5)									9.0	9.0*	108	2.02*	22 to 26							
Average	≈53	≈37	≈29	36.9	19.5	47.1	0.70	1.62	9.3	8.2	133	2.26	all samples	1.476	0.795		0.137	2.4	12.4	26.3

TABLE 5.21 - Summary of data from Field Test nº 6

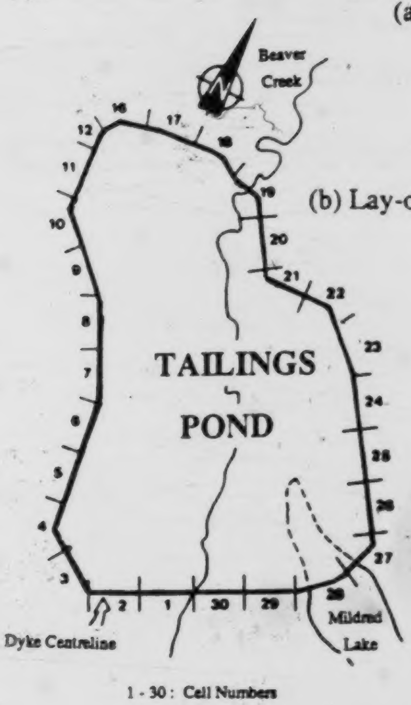
Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
spigot 1				55.8	40.5	27.4														
spigot 4				56.5	39.5	30.1														
prof.I(all sp)				56.2	40.0	28.8			7.0	3.5	278	3.45								
Average				56.2	40.0	28.8														

TABLE 5.22 - Summary of data from Field Test nº 7

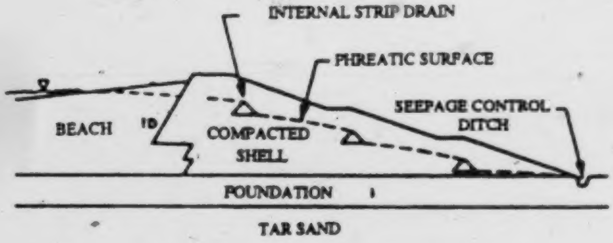
Location	Cw/f A	Cw,s/f	%Ff B	Cw/sp C	Cw,s/sp	%Fsp D	C/A	D/B	Hi(m)	Hf(m)	L(m)	i av(%)	Samples	Density (g/cm3)	Void Ratio	RD (%)	D50 (mm)	CU	%Fb E	FC (%)
spigot 3				66.4	56.9	14.3														
spigot 4				66.9	57.1	14.6														
prof.II(all sp)				66.7	57.0	14.5			8.5	5.0	275	3.09								
Average				66.7	57.0	14.5														

ARCTIC OCEAN
ATLANTIC OCEAN
PACIFIC OCEAN
Edmonton
U.S.A.

(a) Location of tailings dams

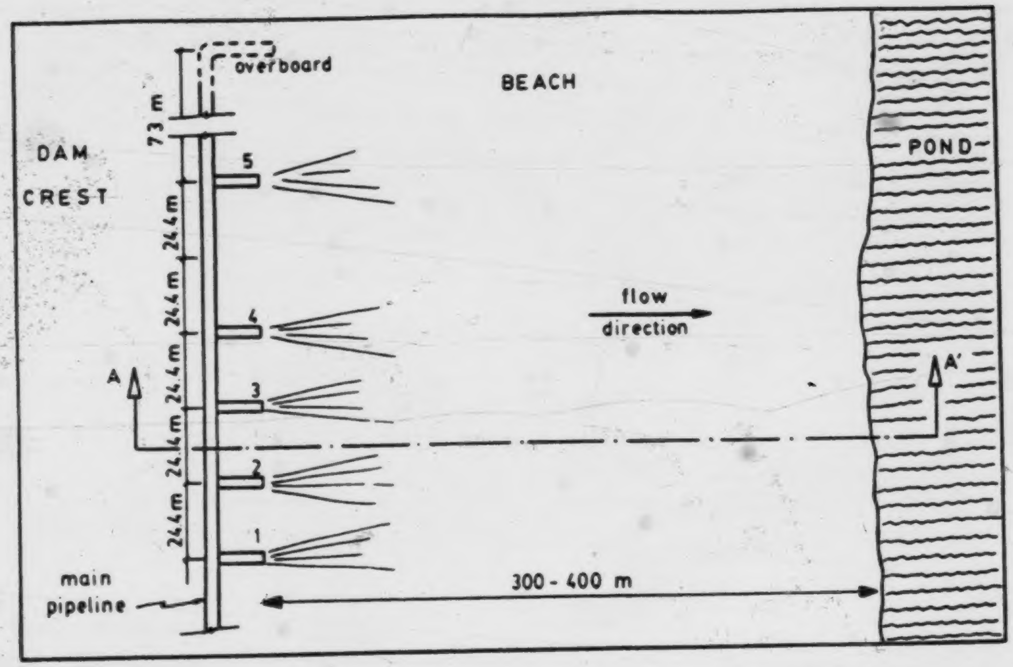

Beaver Creek
12
16
17
18
11
19
10
20
9
21
22
8
23
7
TAILINGS POND
24
6
25
5
26
4
27
3
28
2
1
30
29
Dyke Centreline
Mildred Lake
1 - 30 : Cell Numbers

(b) Lay-out of tailings dam and cell locations

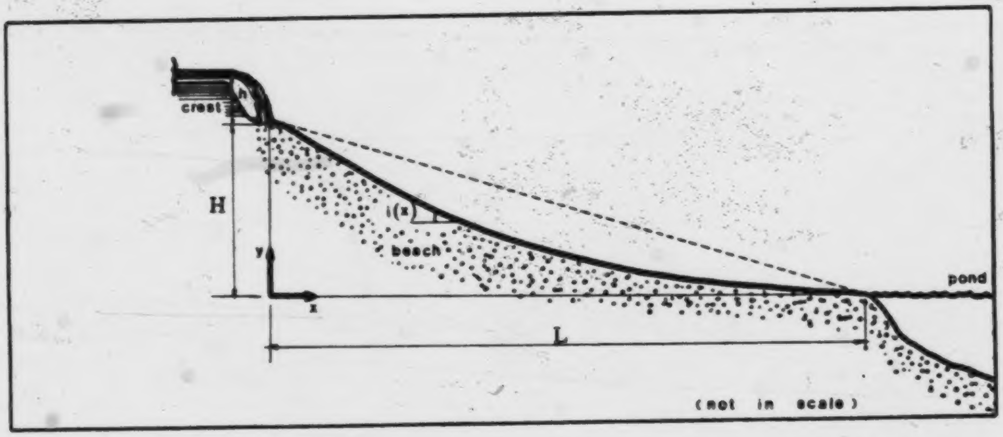

INTERNAL STRIP DRAIN
PHREATIC SURFACE
SEEPAGE CONTROL DITCH
BEACH
10
COMPACTED SHELL
FOUNDATION
TAR SAND

(c)Typical design cross-section

FIG. 5.1 - Site of the field tests - Syncrude's tailings dam

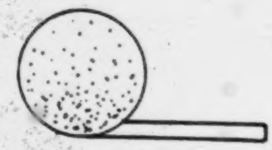

overboard
BEACH
73 m
DAM
CREST
5
24.4m
24.4m
4
flow
direction
24.4m
A
3
A'
POND
24.4m
2
24.4m
1
main
pipeline
300 - 400 m

PLANT

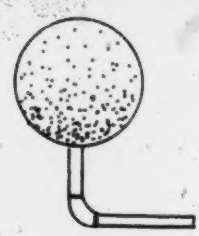

crest
h
H
i(x)
y
beach
pond
x
L
(not in scale)

SECTION A - A'

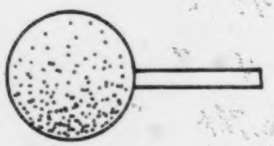
FIG. 5.2 - Lay-out of the field tests



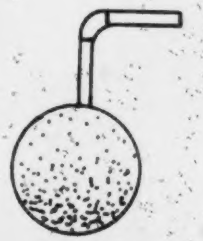
(a) Syncrude's standard spigots (Pilot Test)



(b) Spigots off the bottom of the pipe (Tests 1 and 7)



(c) Spigots off the centre of the pipe (Tests 2, 5 and 6)



(d) Spigots off the top of the pipe (Tests 3 and 4)

FIG. 5.3 - Position of the spigots for the various tests

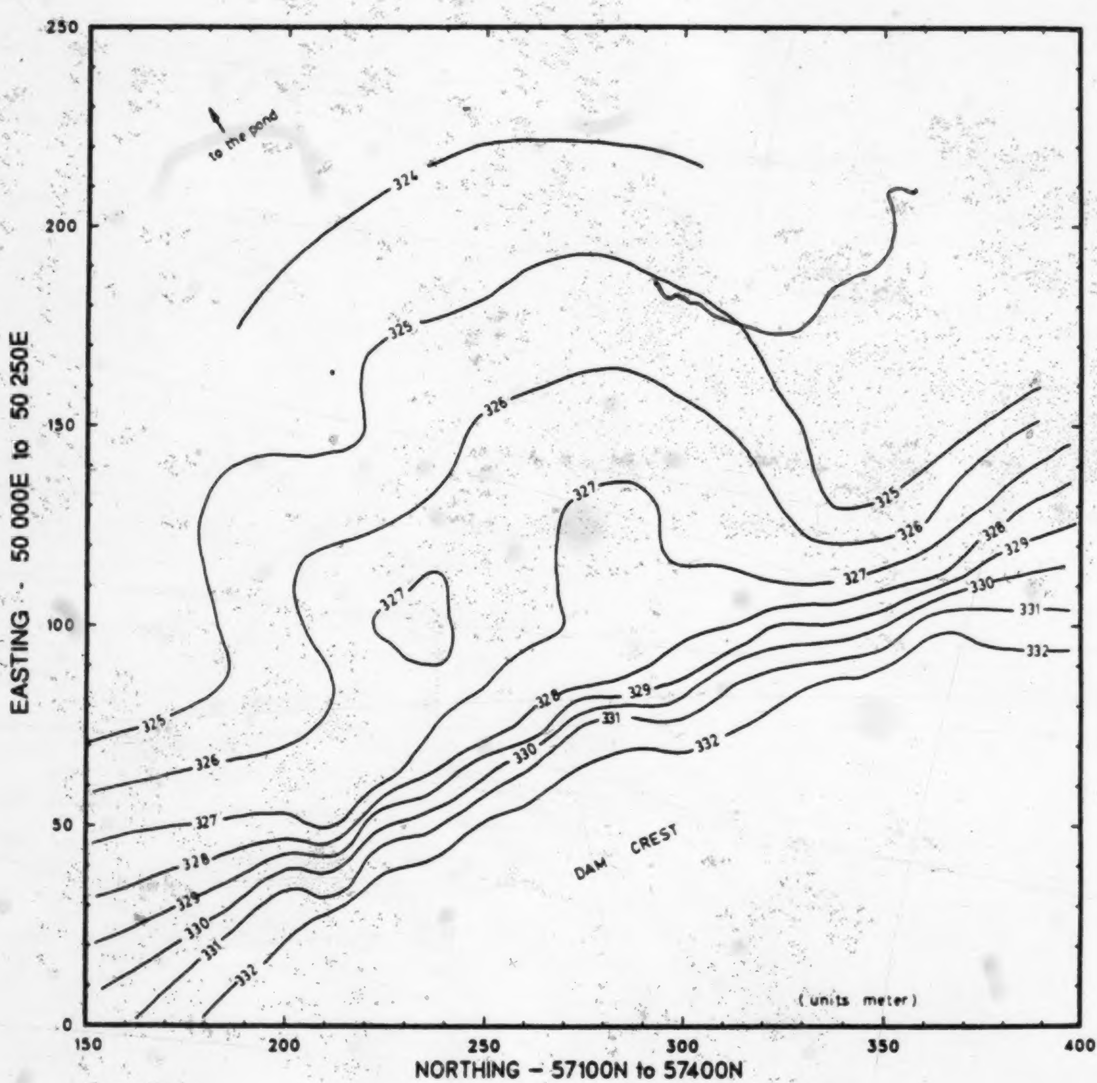

to the pond
324
325
326
327
327
328
329
330
331
332
DAM CREST
(units meter)
EASTING - 50 000E to 50 250E
NORTHING - 57100N to 57400N
0
50
100
150
200
250
150
200
250
300
350
400

Fig. 5.4 - Contour map of the beach before the pilot test (Test nº 0)

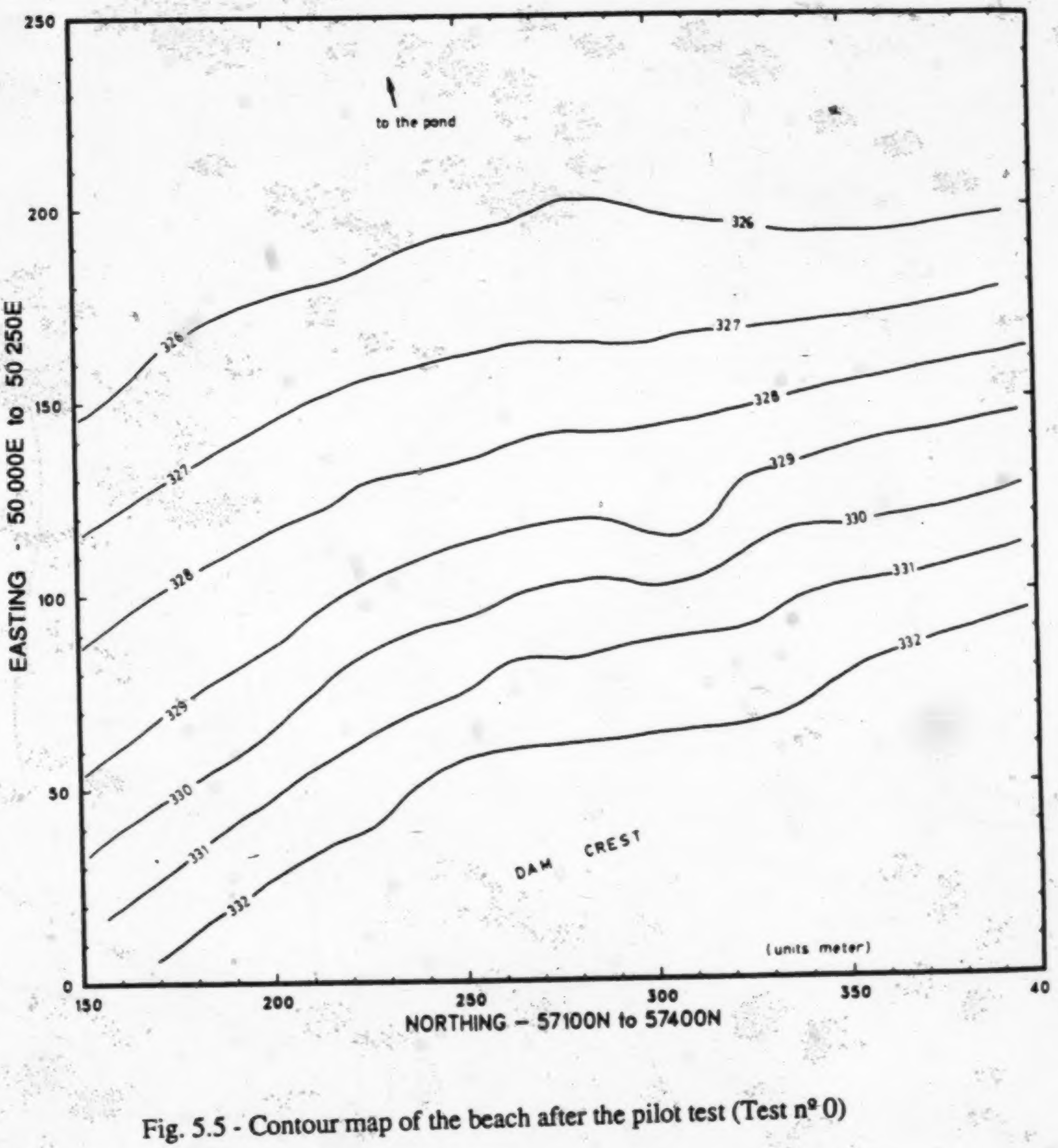

to the pond
326
326
327
327
328
328
329
329
330
330
331
331
332
332
DAM CREST
(units meter)
EASTING - 50 000E to 50 250E
250
200
150
100
50
0
150
200
250
300
350
40
NORTHING — 57100N to 57400N

Fig. 5.5 - Contour map of the beach after the pilot test (Test nº 0)

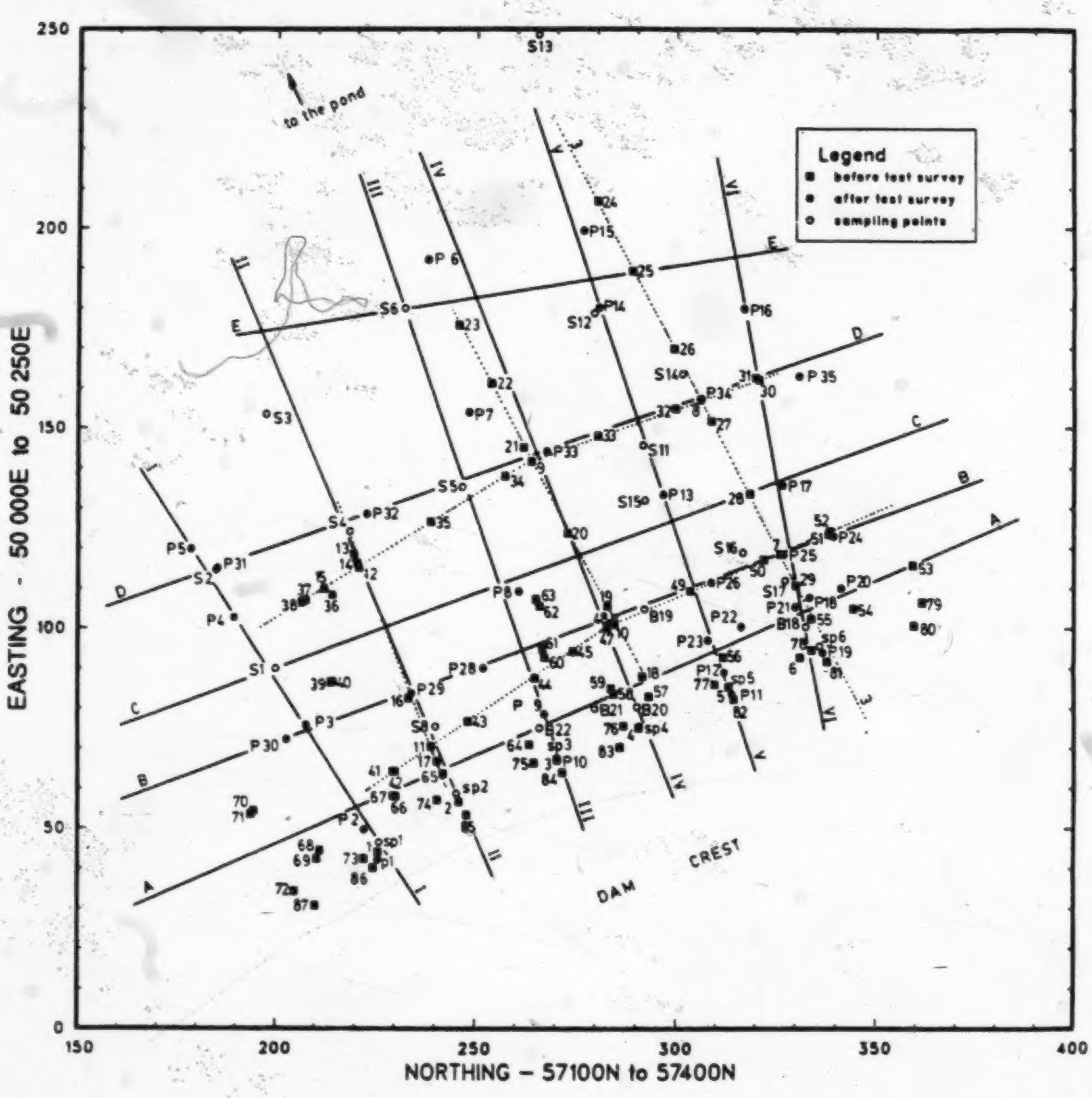

EASTING - 50 000E to 50 250E
NORTHING - 57100N to 57400N
0
50
100
150
200
250
150
200
250
300
350
400
to the pond
Legend
■ before test survey
● after test survey
○ sampling points
A
B
C
D
E
I
II
III
IV
V
VI
DAM CREST
S1
S2
S3
S4
S5
S6
S8
S11
S12
S13
S14
S15
S16
S17
P2
P3
P4
P5
P6
P7
P8
P13
P14
P15
P16
P17
P18
P19
P20
P21
P22
P23
P24
P25
P26
P28
P29
P30
P31
P32
P33
P34
P35
sp1
sp2
sp3
sp4
sp5
sp6

Fig. 5.6 - Location of cross-sections and sampling points - Test nº 0

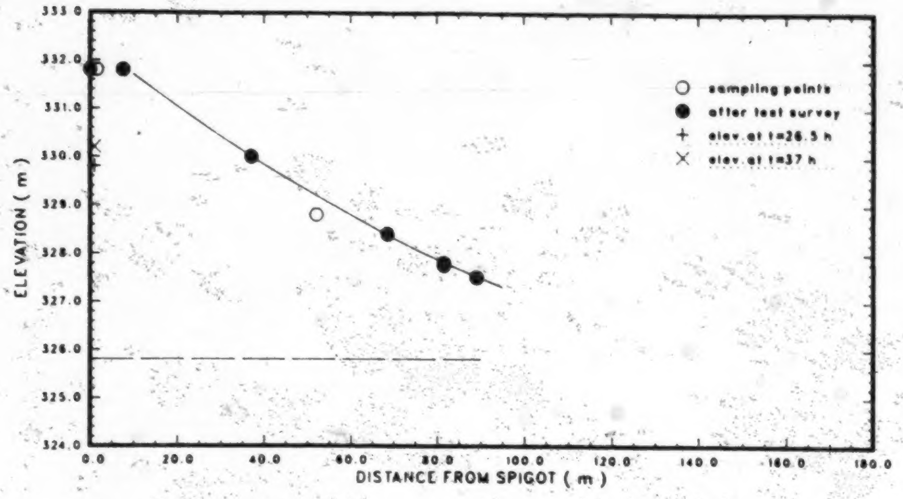


FIG. 5.7 - Profile I - Spigot 1 - Test nº 0

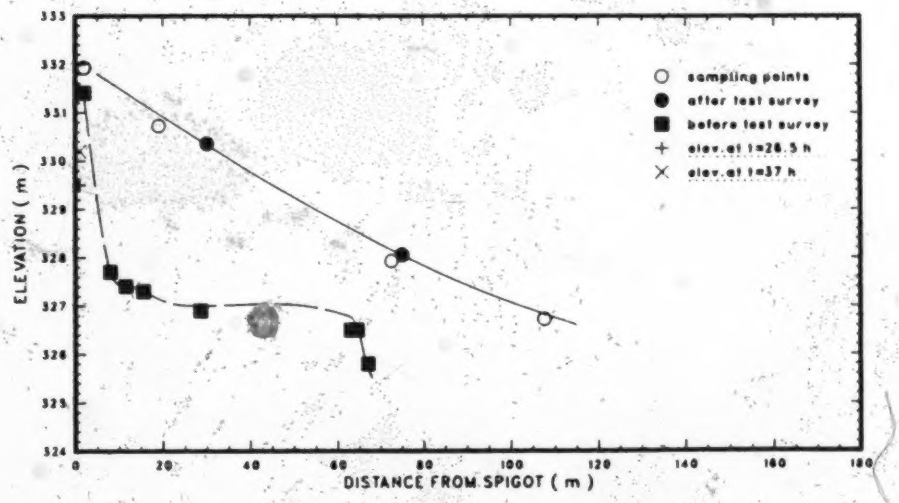


FIG. 5.8 - Profile II - Spigot 2 - Test nº 0

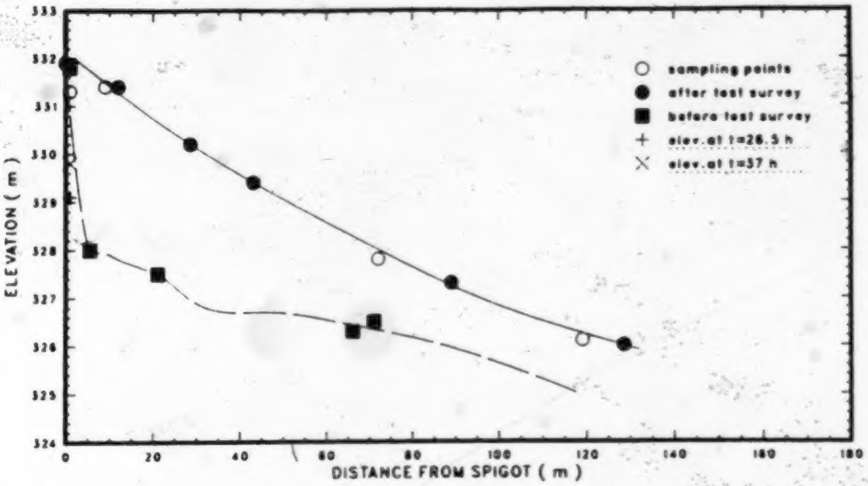


FIG. 5.9 - Profile III - Spigot 3 - Test nº 0

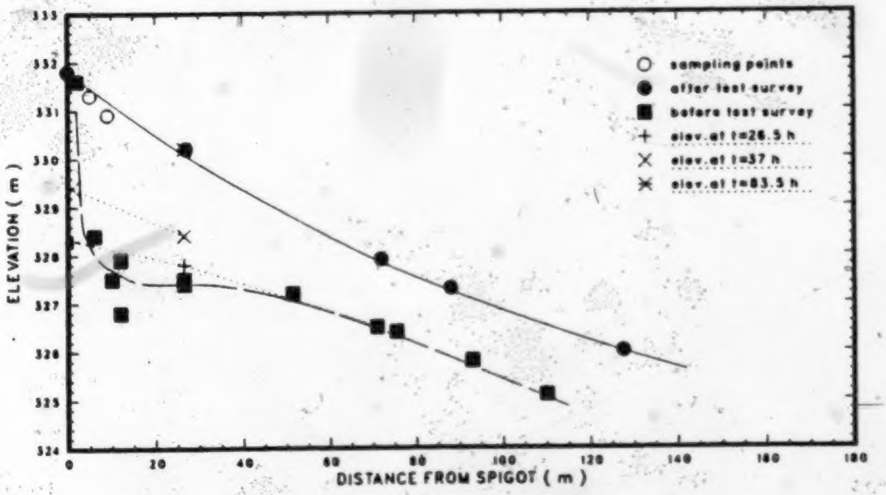


FIG. 5.10 - Profile IV - Spigot 4 - Test nº 0

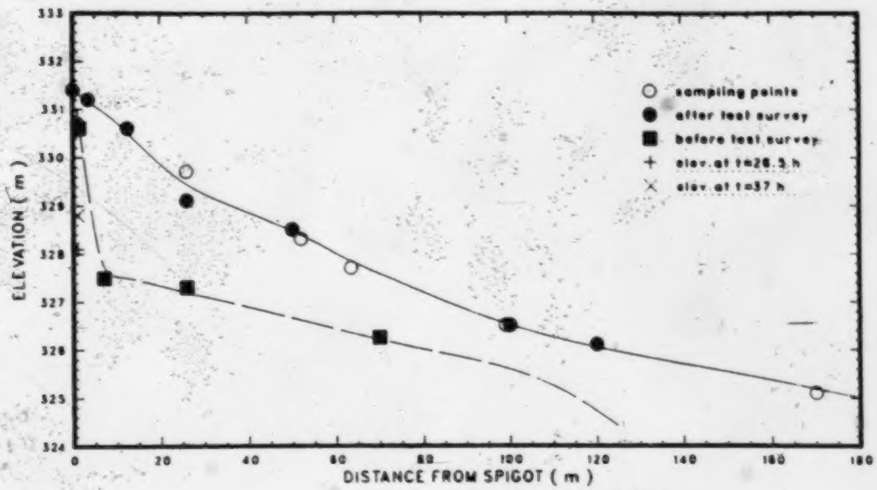


FIG. 5.11 - Profile V - Spigot 5 - Test nº 0

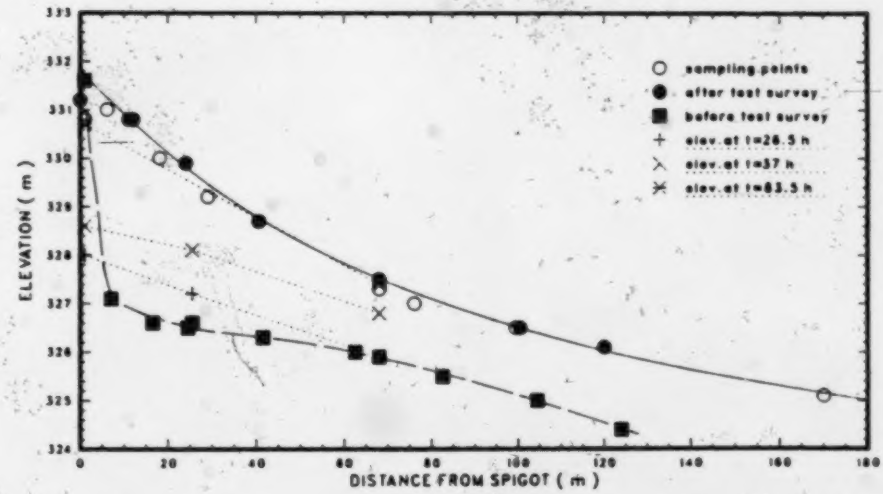


FIG. 5.12 - Profile 3 - Spigots 5/6 - Test nº 0

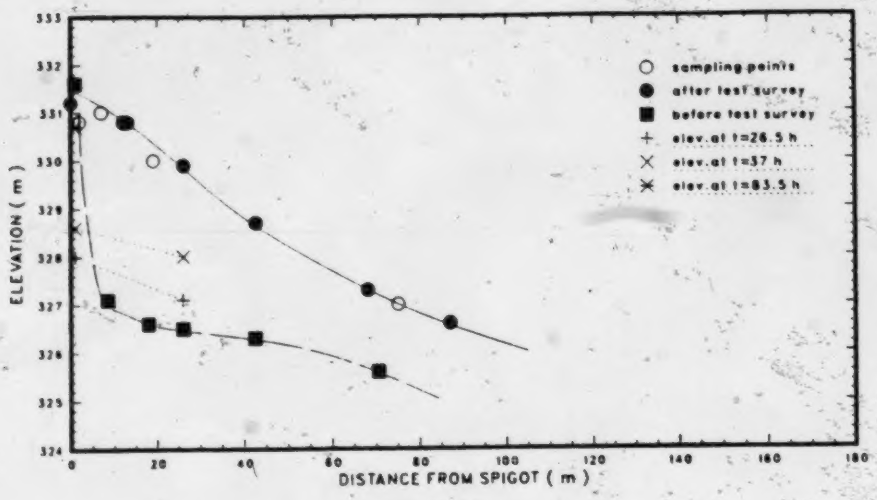


FIG. 5.13 - Profile VI - Spigot 6 - Test nº 0

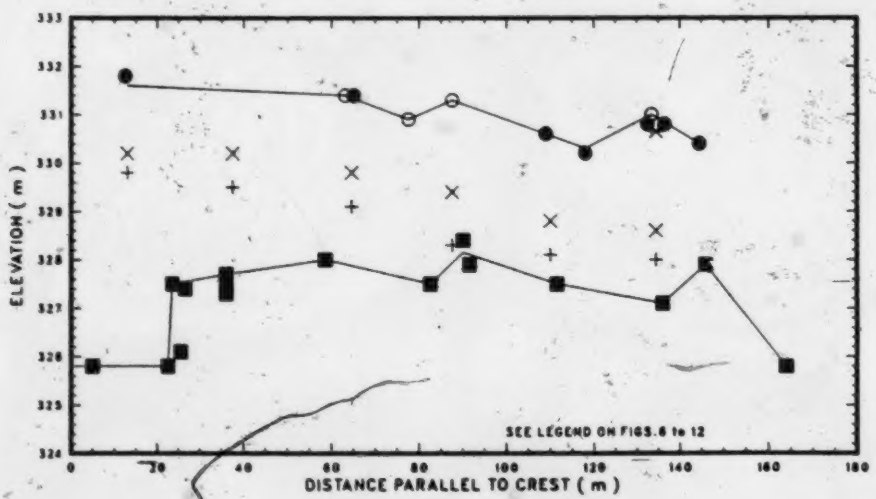


FIG. 5.14 - Profile A - Test nº 0

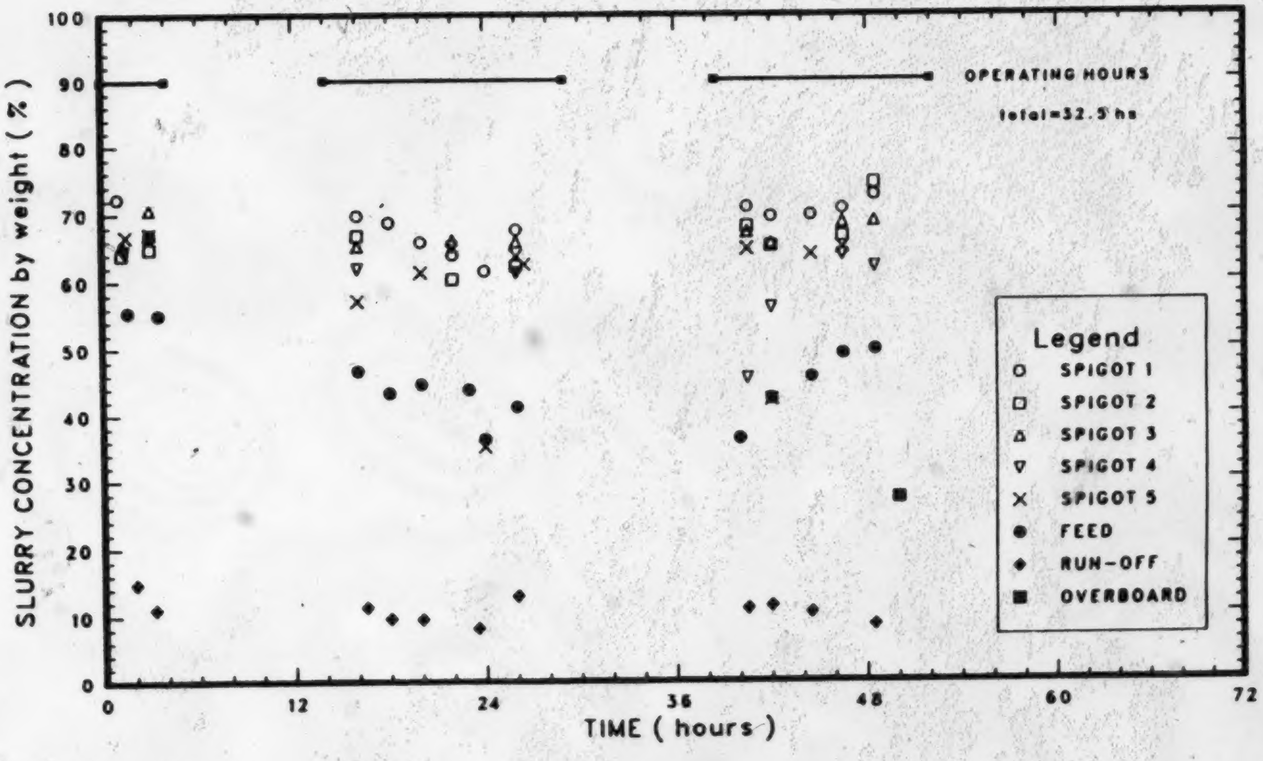


FIG. 5.15 - Variation of the input parameters with time - Test nº 1

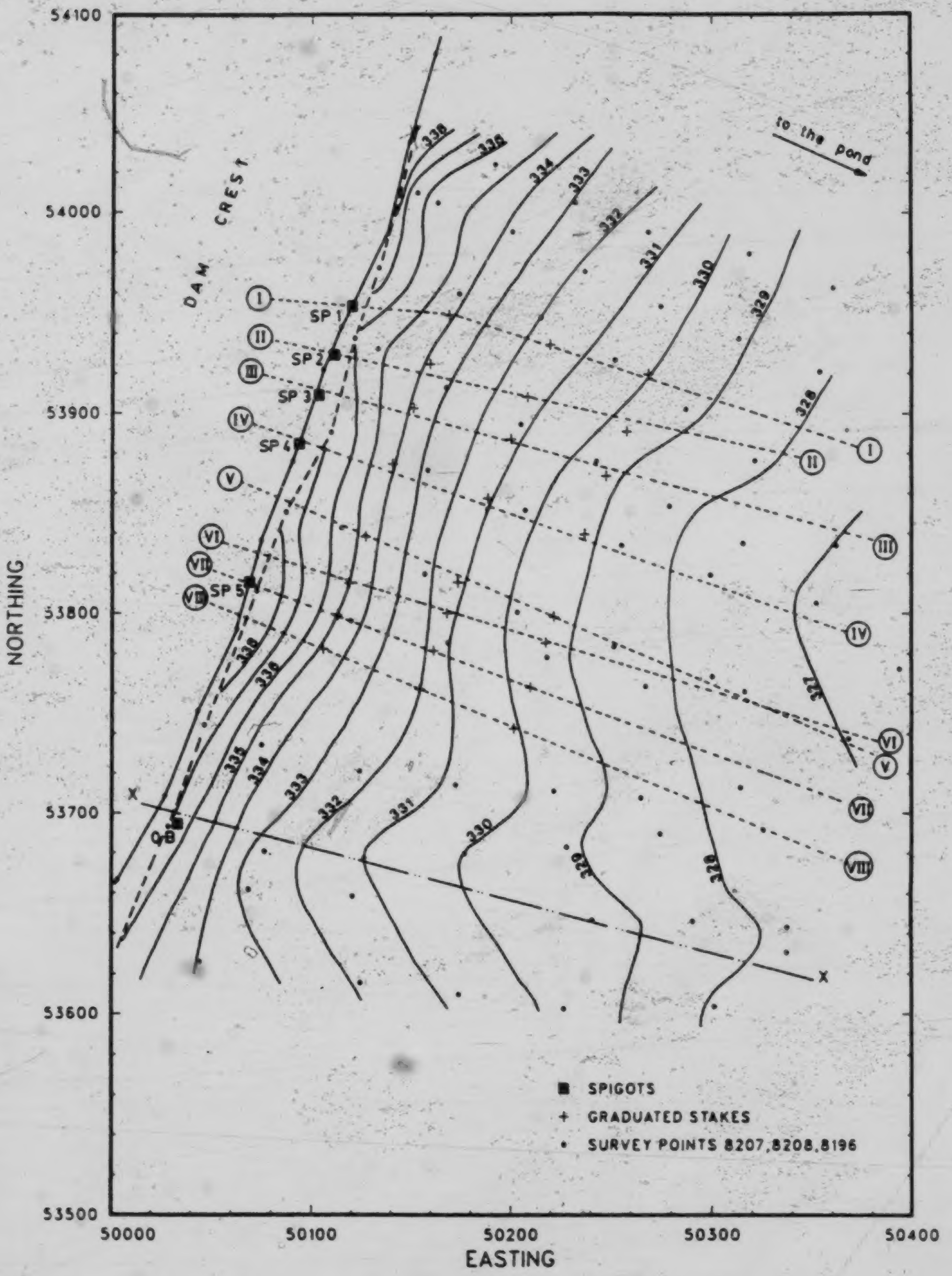
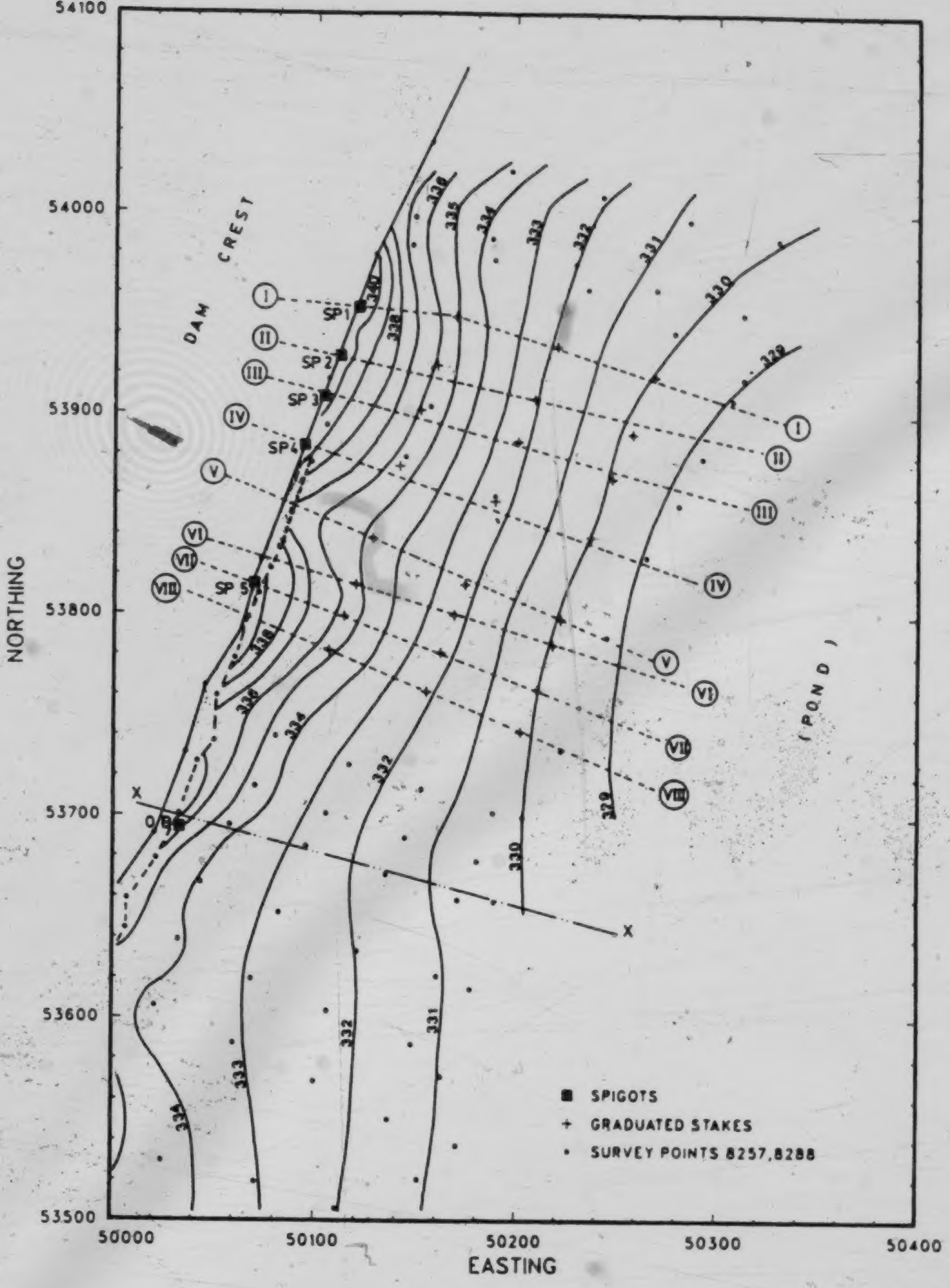

54100
54000
53900
53800
53700
53600
53500
NORTHING
50000
50100
50200
50300
50400
EASTING
DAM CREST
to the pond
SP 1
SP 2
SP 3
SP 4
SP 5
O.B
I
II
III
IV
V
VI
VII
VIII
338
336
335
334
333
332
331
330
329
328
327
X
X
■ SPIGOTS
+ GRADUATED STAKES
• SURVEY POINTS 8207,8208,8196

Fig. 5.16 - Contour map of the beach before Test nº 1



54100
54000
53900
53800
53700
53600
53500
NORTHING
50000
50100
50200
50300
50400
EASTING
DAM CREST
(POND)
SP1
SP2
SP3
SP4
SP5
I
II
III
IV
V
VI
VII
VIII
X
X
O/B
340
338
336
335
334
333
332
331
330
329
■ SPIGOTS
+ GRADUATED STAKES
• SURVEY POINTS 8257,8288

Fig. 5.17 - Contour map of the beach after Test nº 1

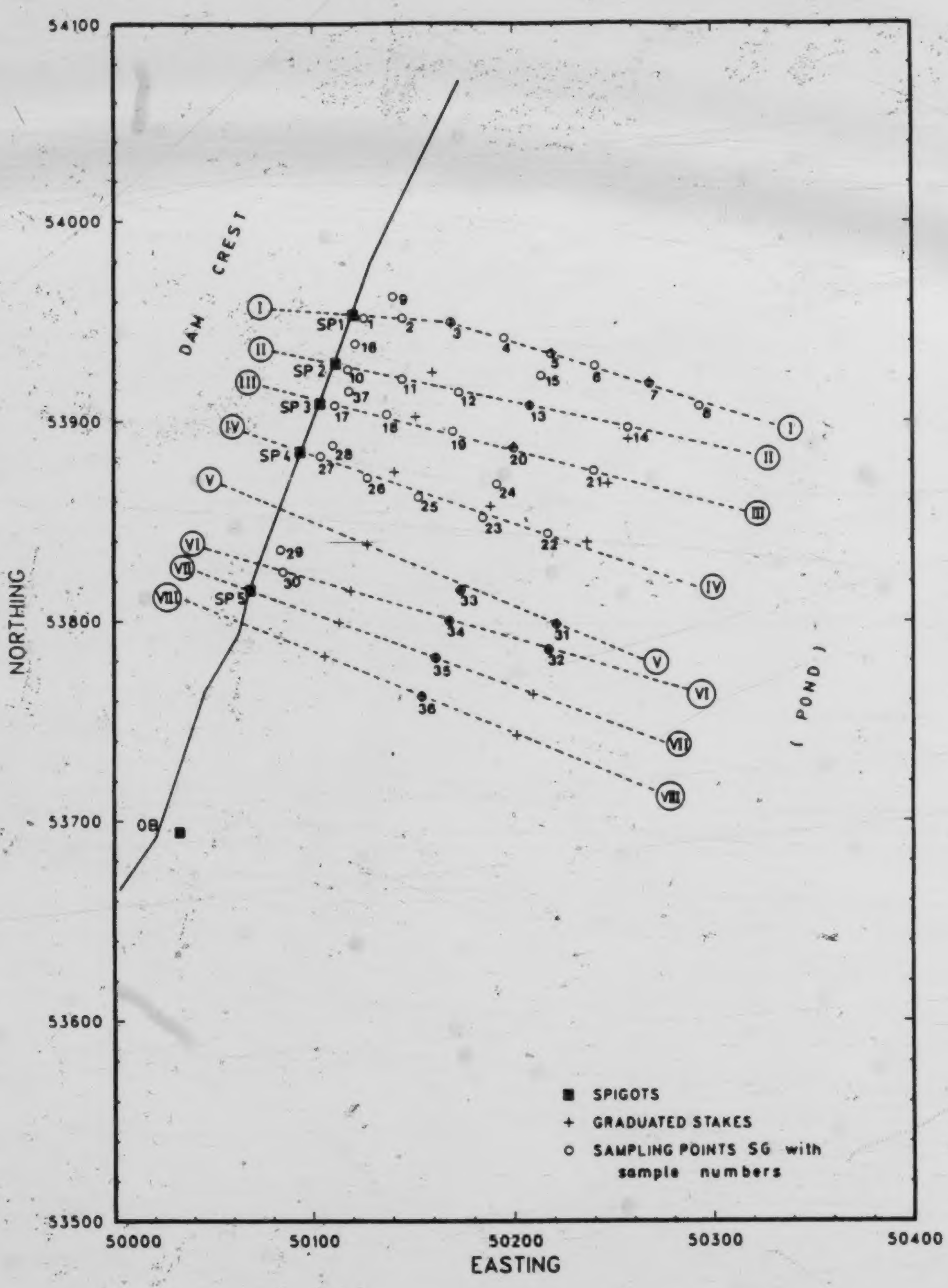

54100
54000
53900
53800
53700
53600
53500
50000
50100
50200
50300
50400
NORTHING
EASTING
DAM CREST
(POND)
SP1
SP 2
SP 3
SP4
SP5
OB
I
II
III
IV
V
VI
VII
VIII
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
■ SPIGOTS
+ GRADUATED STAKES
○ SAMPLING POINTS SG with sample numbers

Fig. 5.18 - Location of cross-sections and sampling points - Test nº 1

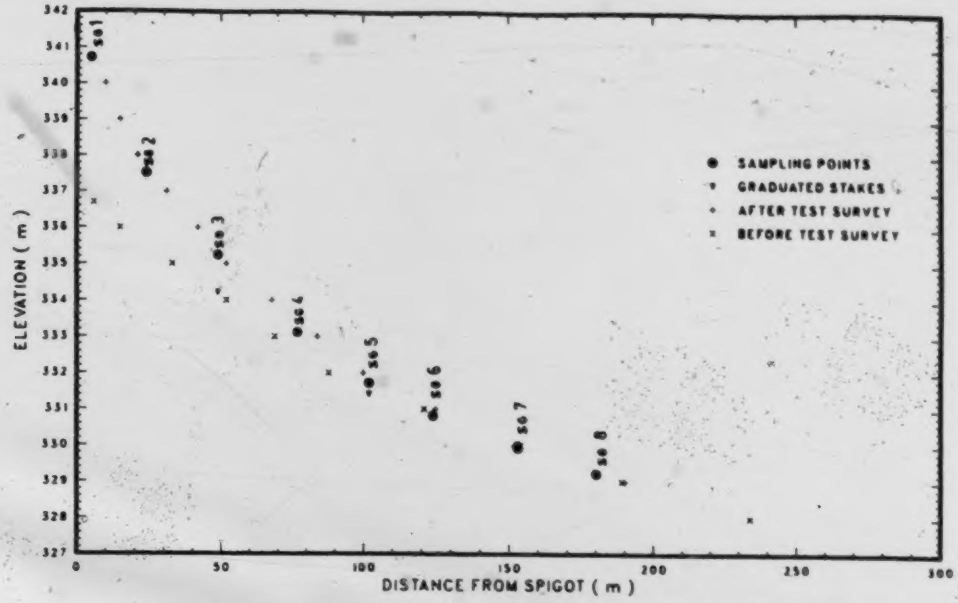


FIG. 5.19 - Profile I - Spigot 1 - Test nº 1

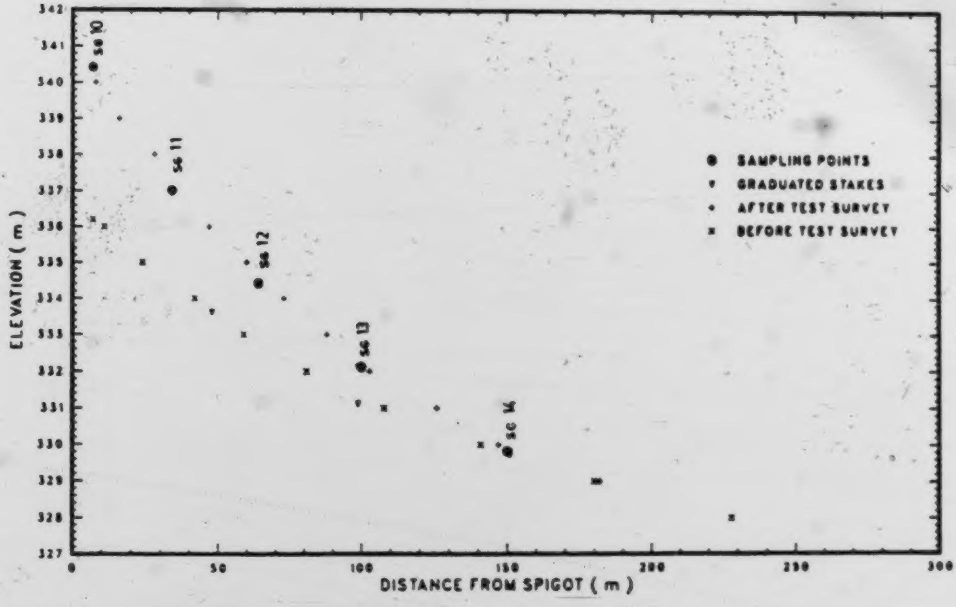


FIG. 5.20 - Profile II - Spigot 2 - Test nº 1

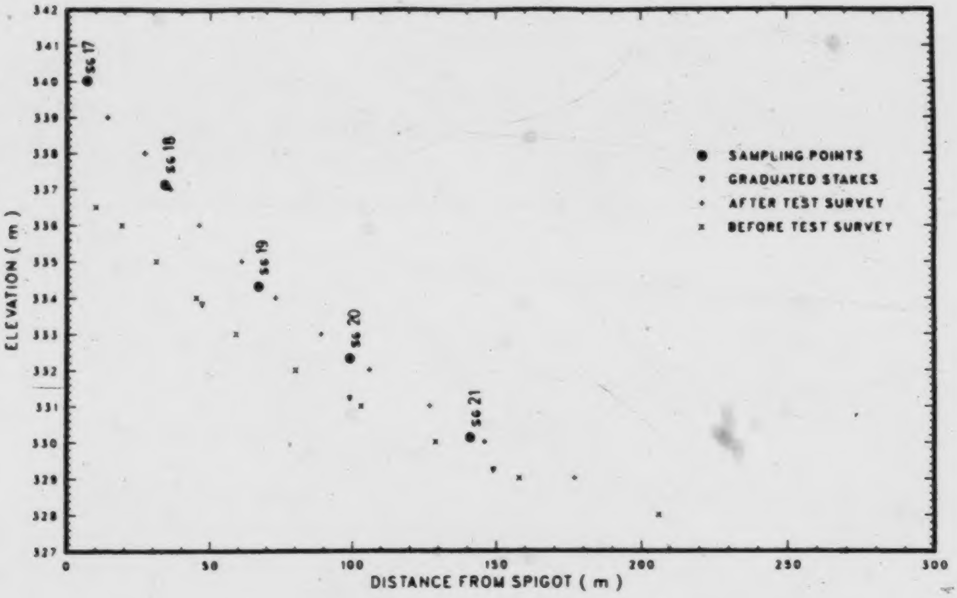


FIG. 5.21 - Profile III - Spigot 3 - Test nº 1

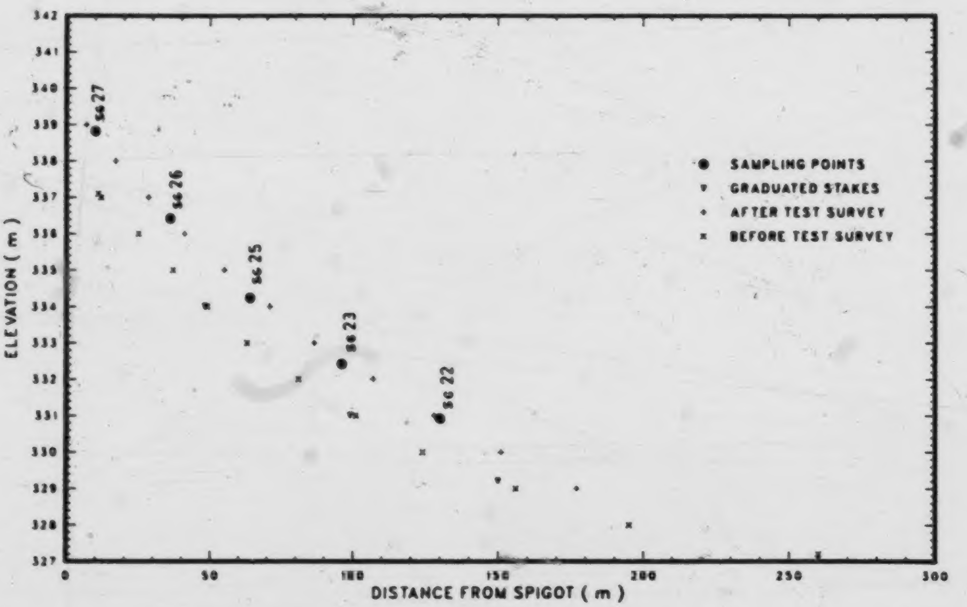


FIG. 5.22 - Profile IV - Spigot 4 - Test nº 1

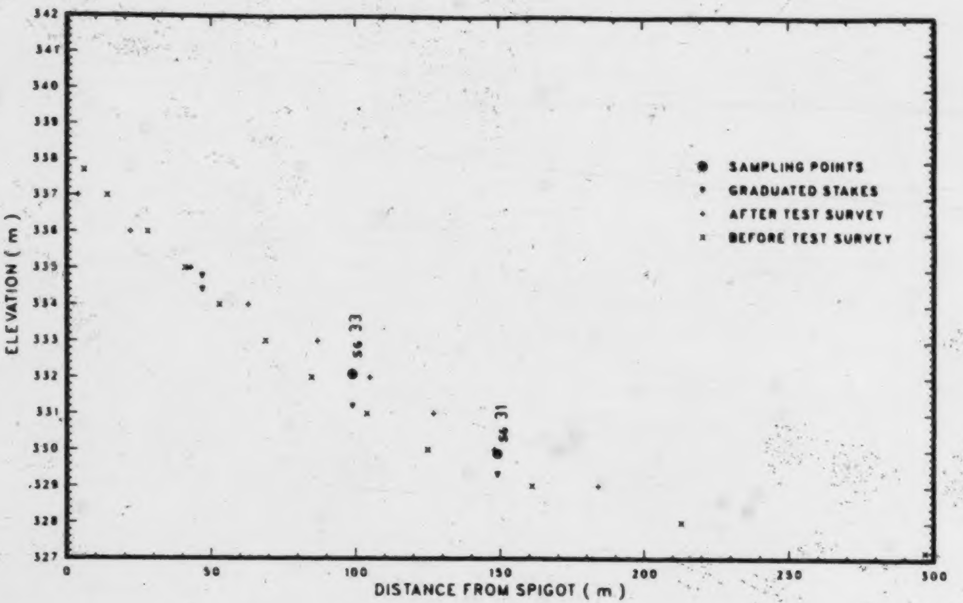


FIG. 5.23 - Profile V - Between spigots 4 and 5 - Test nº 1

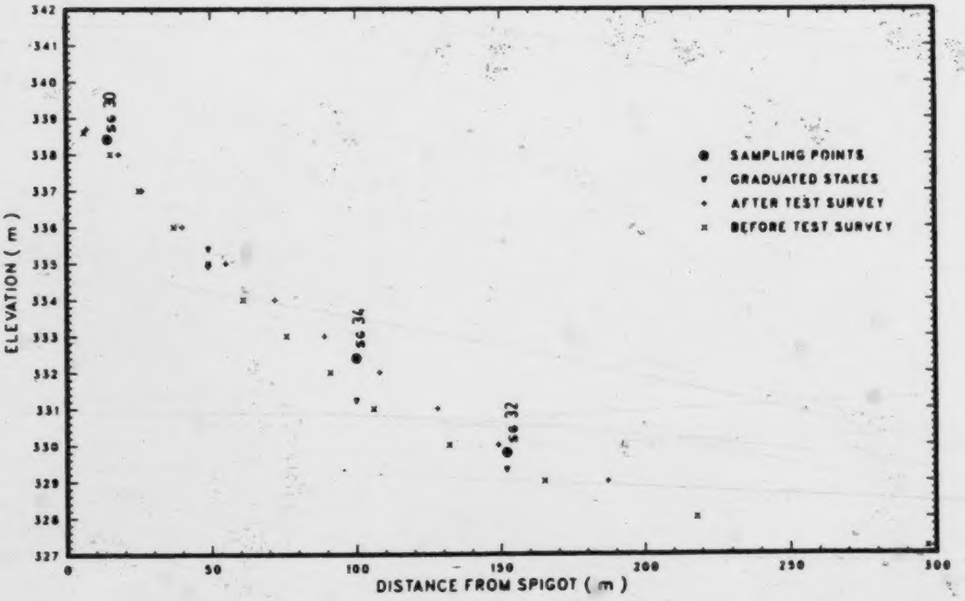


FIG. 5.24 - Profile VI - Spigots 5 (channel) - Test nº 1

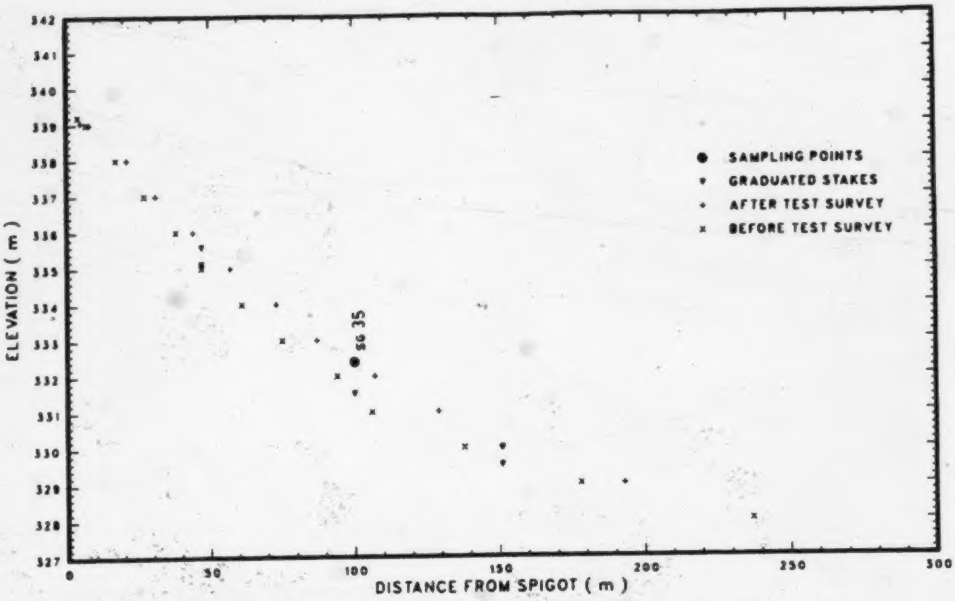


FIG. 5.25 - Profile VII - Spigot 5 - Test nº 1

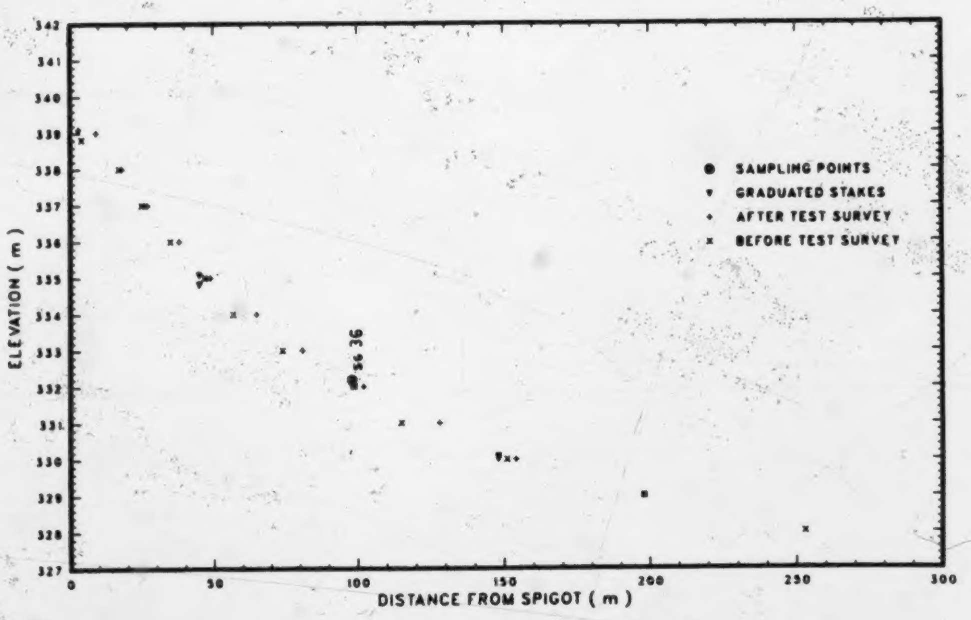


FIG. 5.26 - Profile VIII - Test nº 1

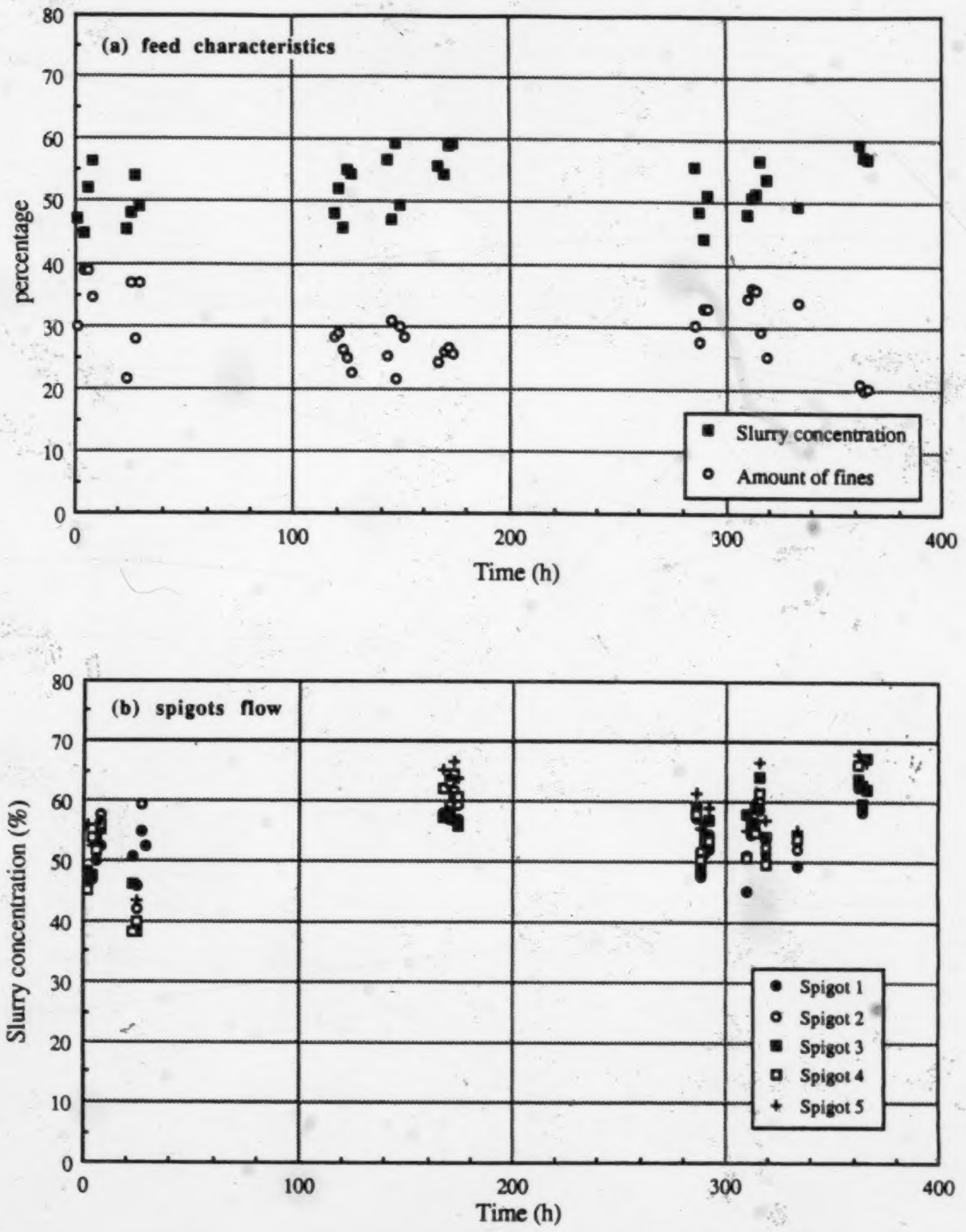
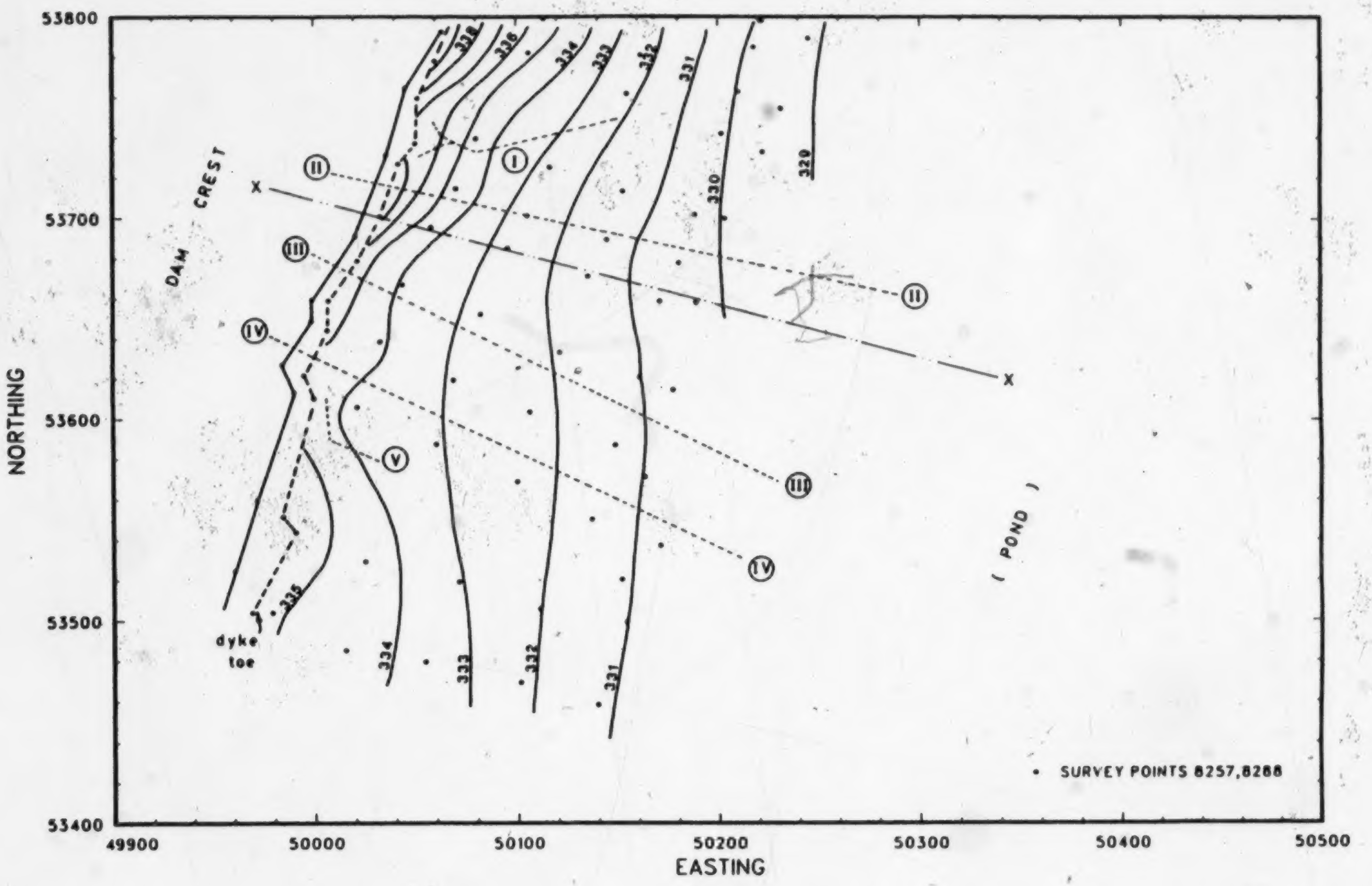


FIG. 5.27 - Variation of the input parameters with time - Field Test nº 2



53800
53700
53600
53500
53400
NORTHING
49900
50000
50100
50200
50300
50400
50500
EASTING
DAM CREST
dyke toe
(POND)
• SURVEY POINTS 8257,8288
338
336
334
333
332
331
330
329
335
334
333
332
331
I
II
III
IV
V
X

Fig. 5.28 - Contour map of the beach before Test nº 2

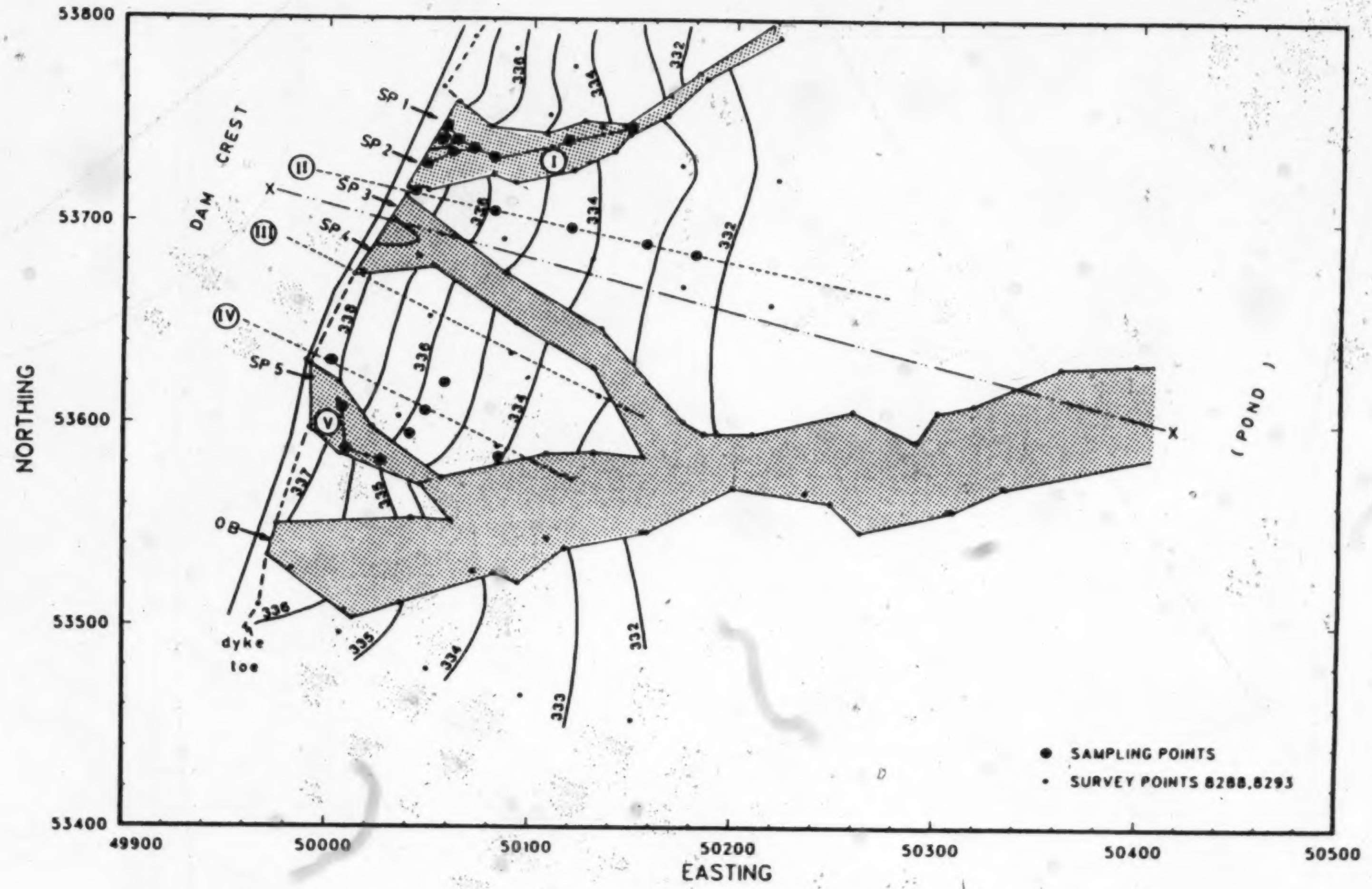

NORTHING
53800
53700
53600
53500
53400
EASTING
49900
50000
50100
50200
50300
50400
50500
DAM CREST
SP 1
SP 2
SP 3
SP 4
SP 5
OB
dyke toe
I
II
III
IV
V
X
X
336
334
332
338
337
335
333
(POND)
● SAMPLING POINTS
• SURVEY POINTS 8288,8293

Fig. 5.29 - Contour map of the beach after Test nº 2

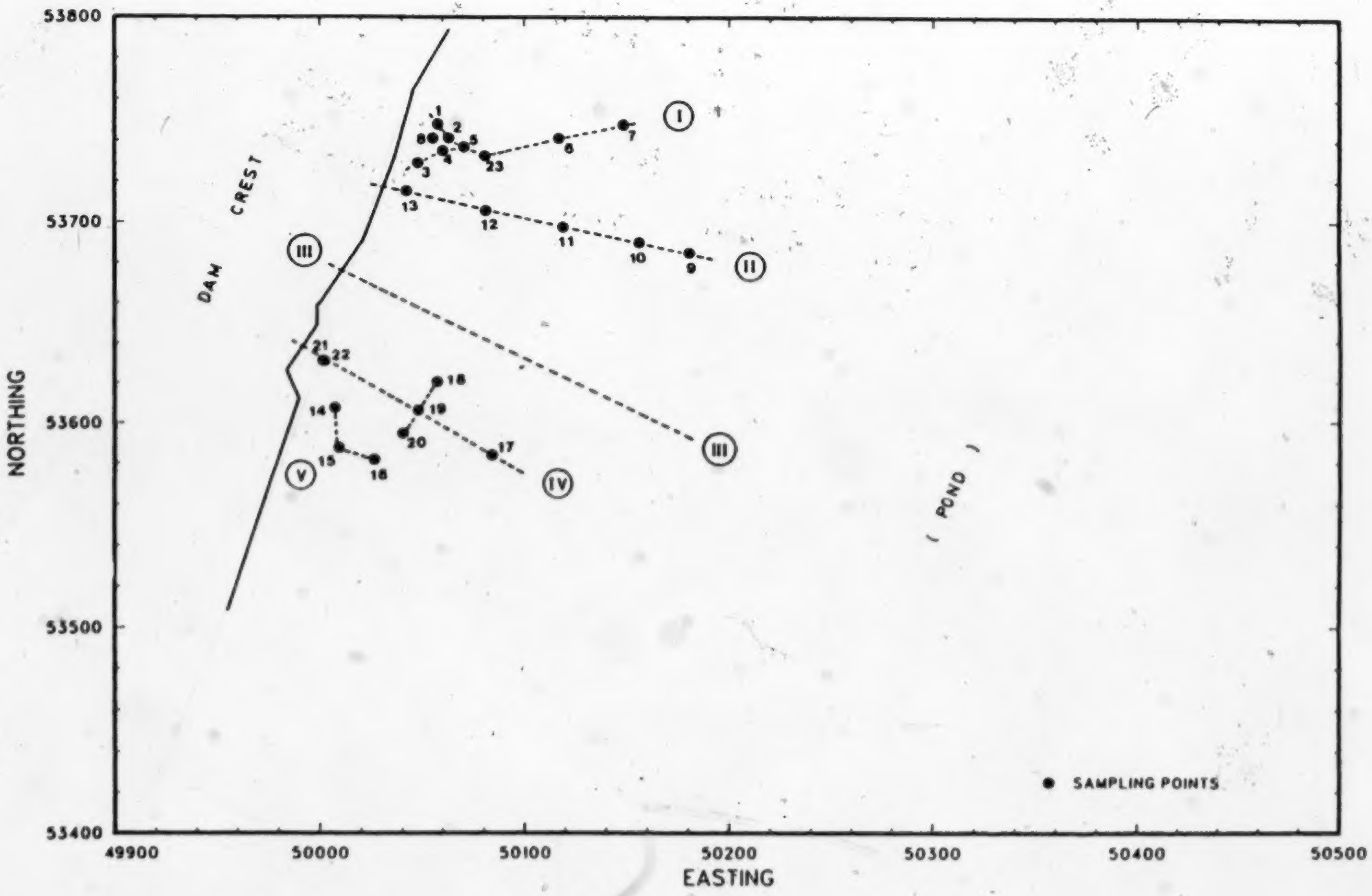

53800
53700
53600
53500
53400
NORTHING
49900
50000
50100
50200
50300
50400
50500
EASTING
DAM CREST
(POND)
I
II
III
III
IV
V
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
● SAMPLING POINTS

Fig. 5.30 - Location of cross-sections and sampling points - Test nº 2

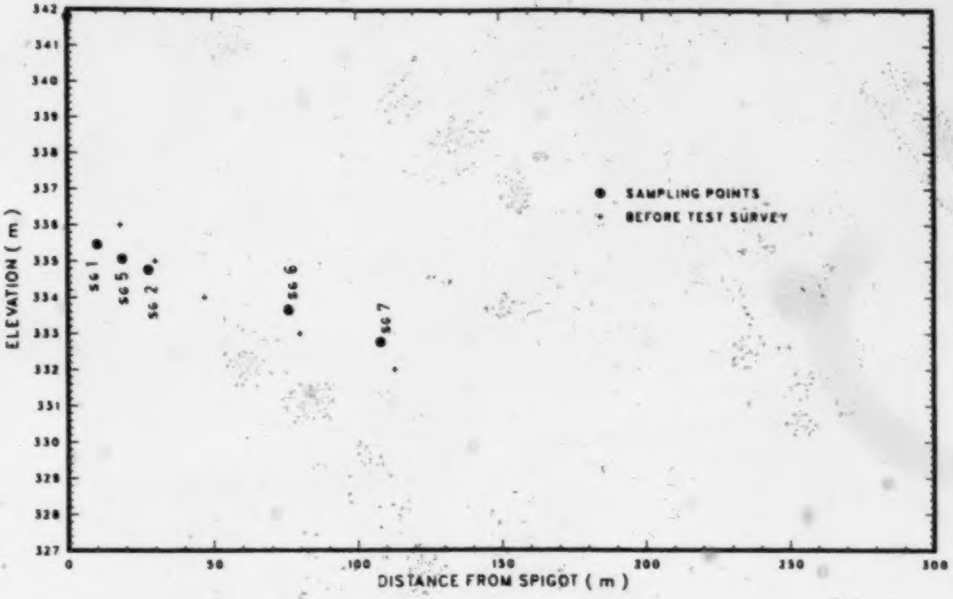


FIG. 5.31 - Profile I - Spigot 1 - Test nº 2

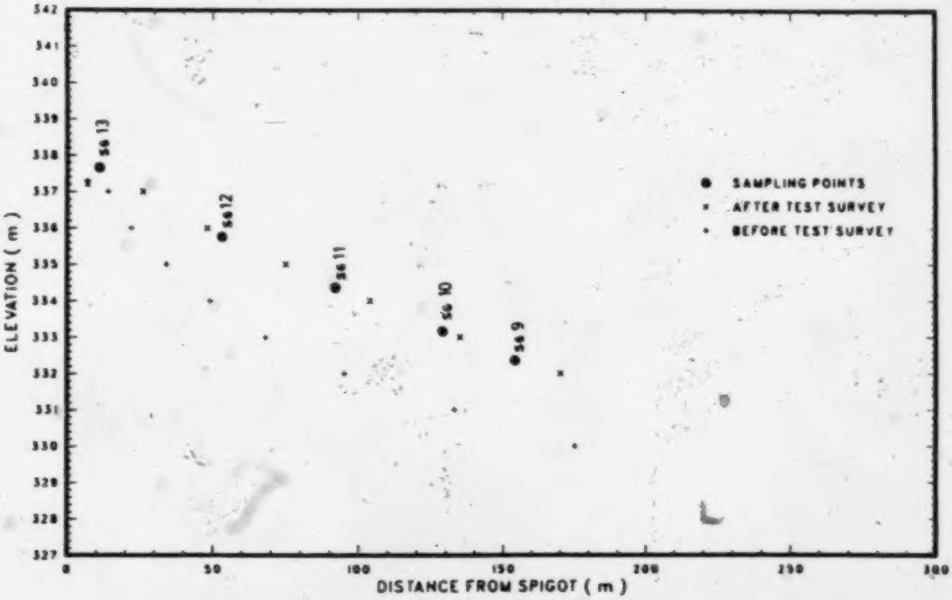


FIG. 5.32 - Profile II - Test nº 2

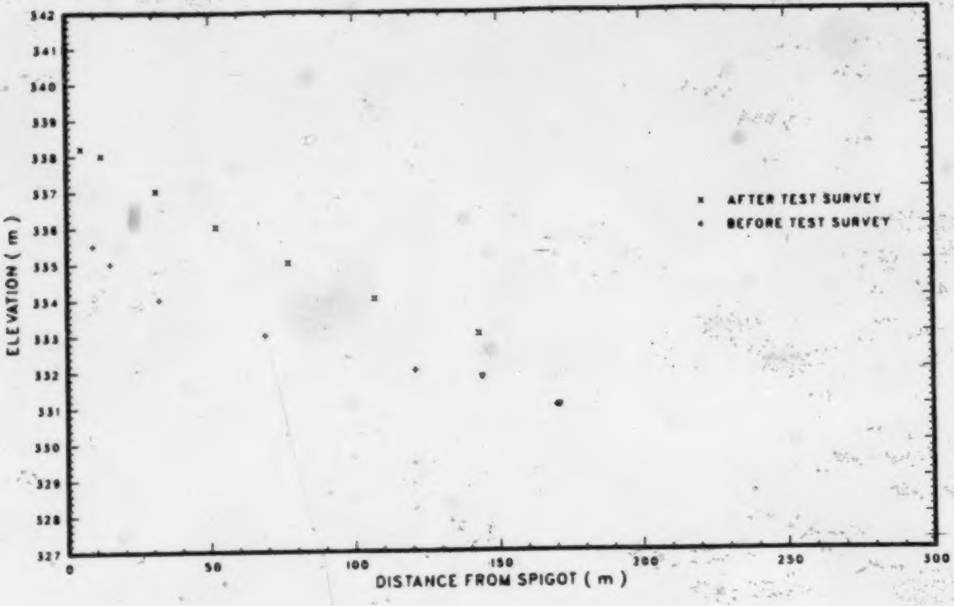


FIG. 5.33 - Profile III - Test nº 2

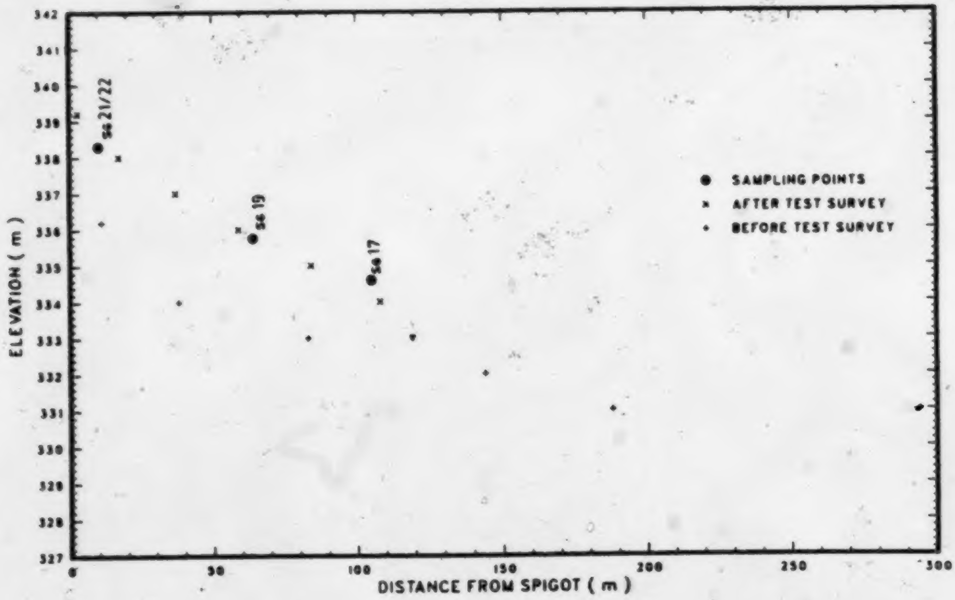


FIG. 5.34 - Profile IV - Test nº 2

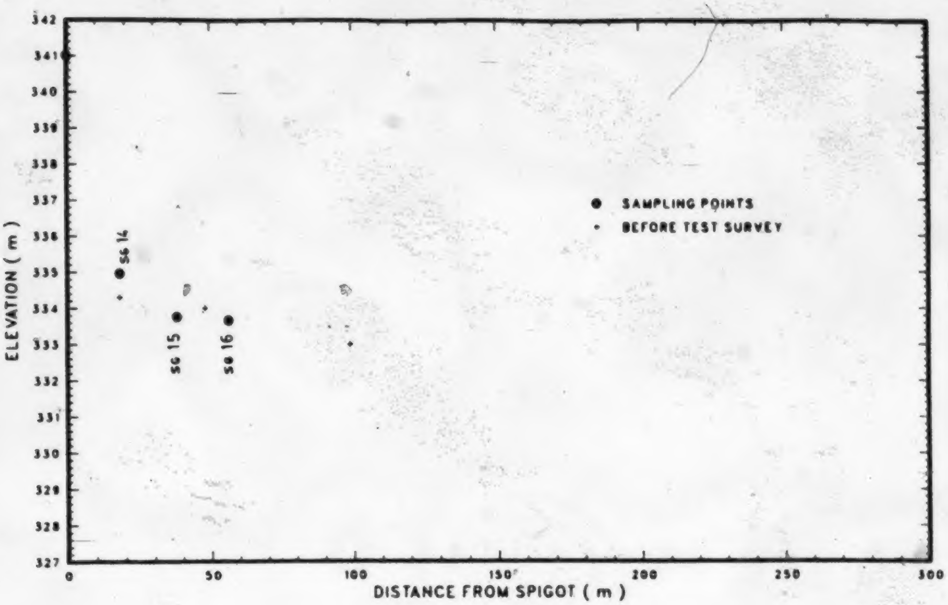


FIG. 5.35 - Profile V - Spigot 5 - Test nº 2

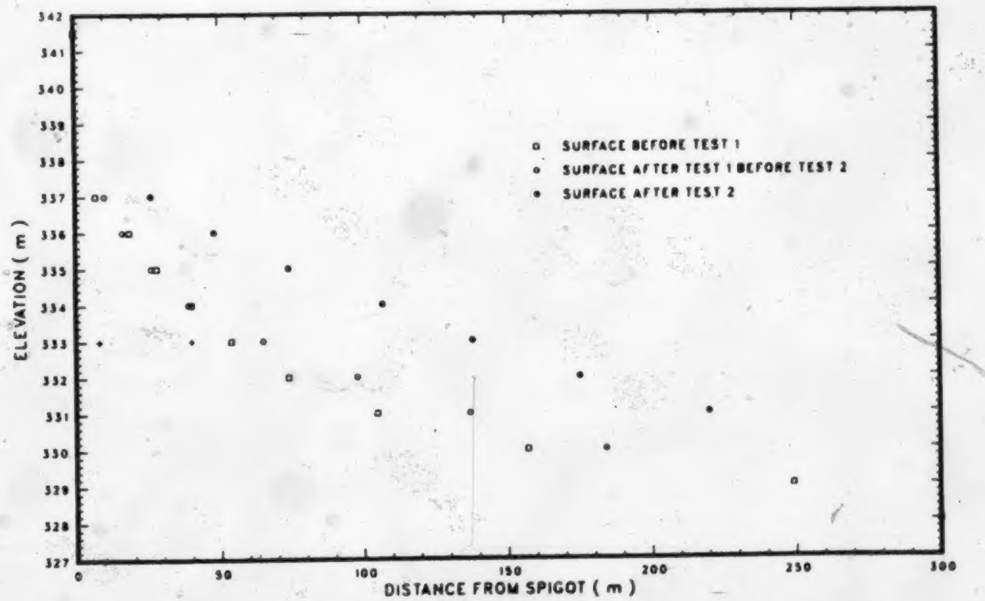
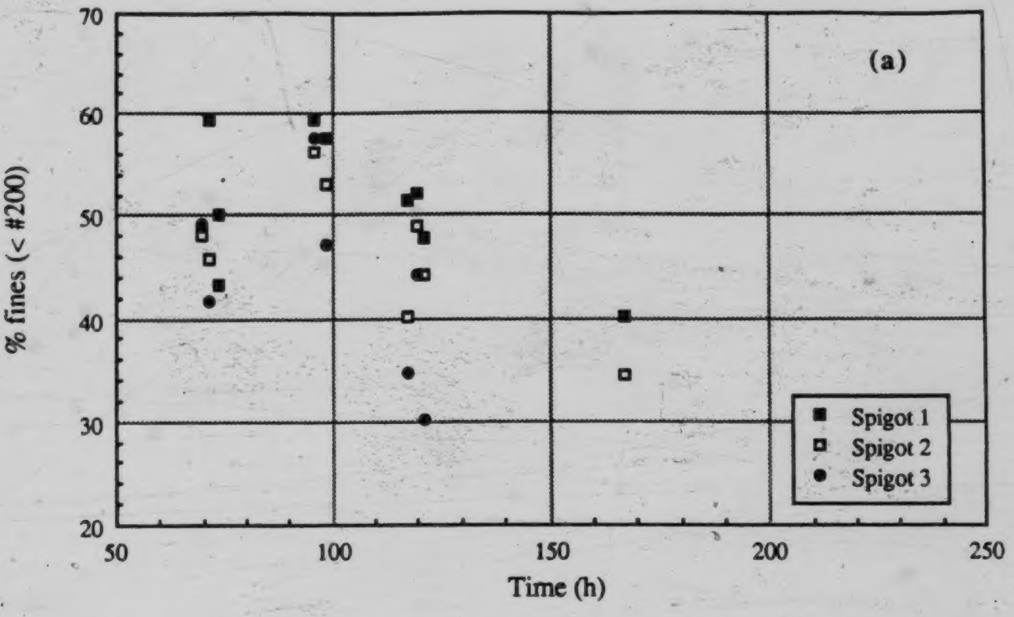


FIG. 5.36 - Profile X - Area of Test nº 1 and Test nº 2



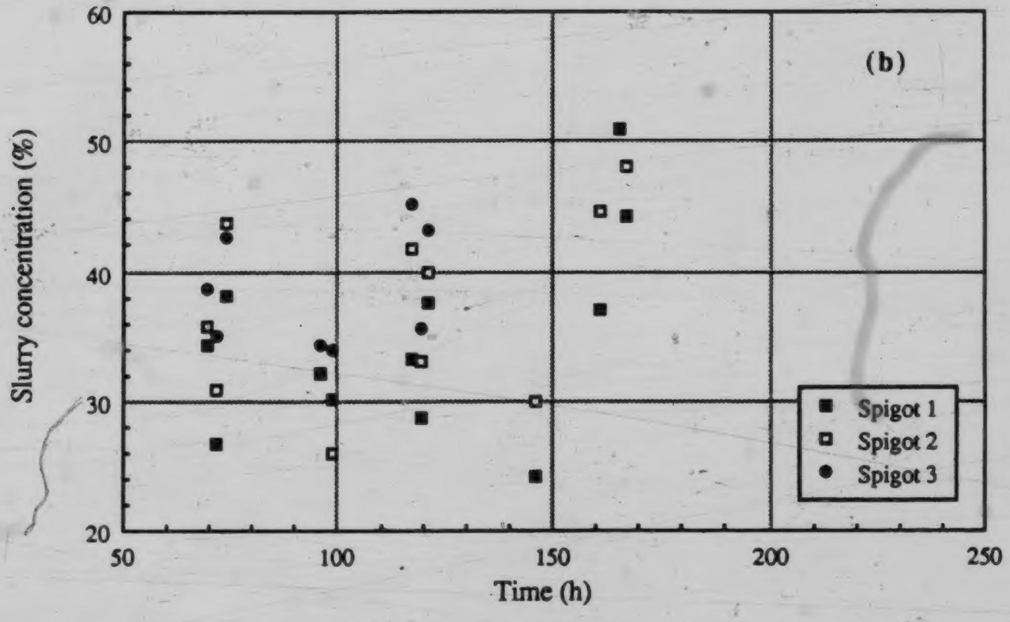


FIG. 5.37 - Variation of input parameters with time - Field Test nº 4

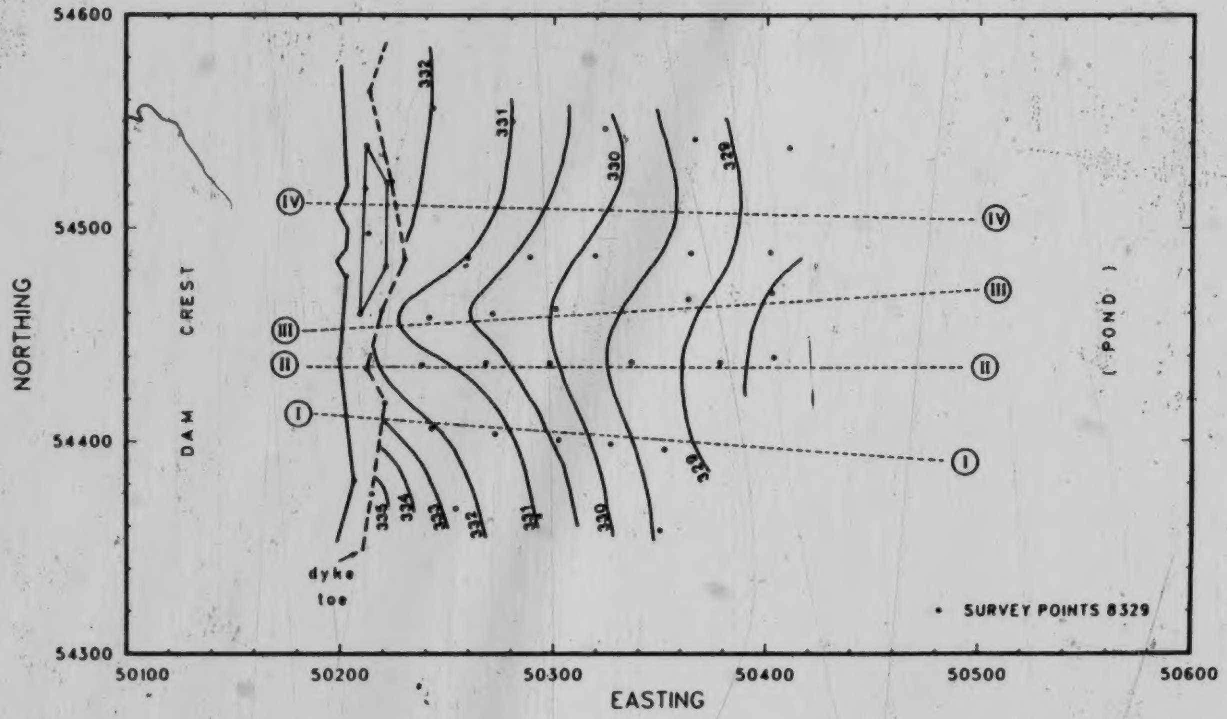
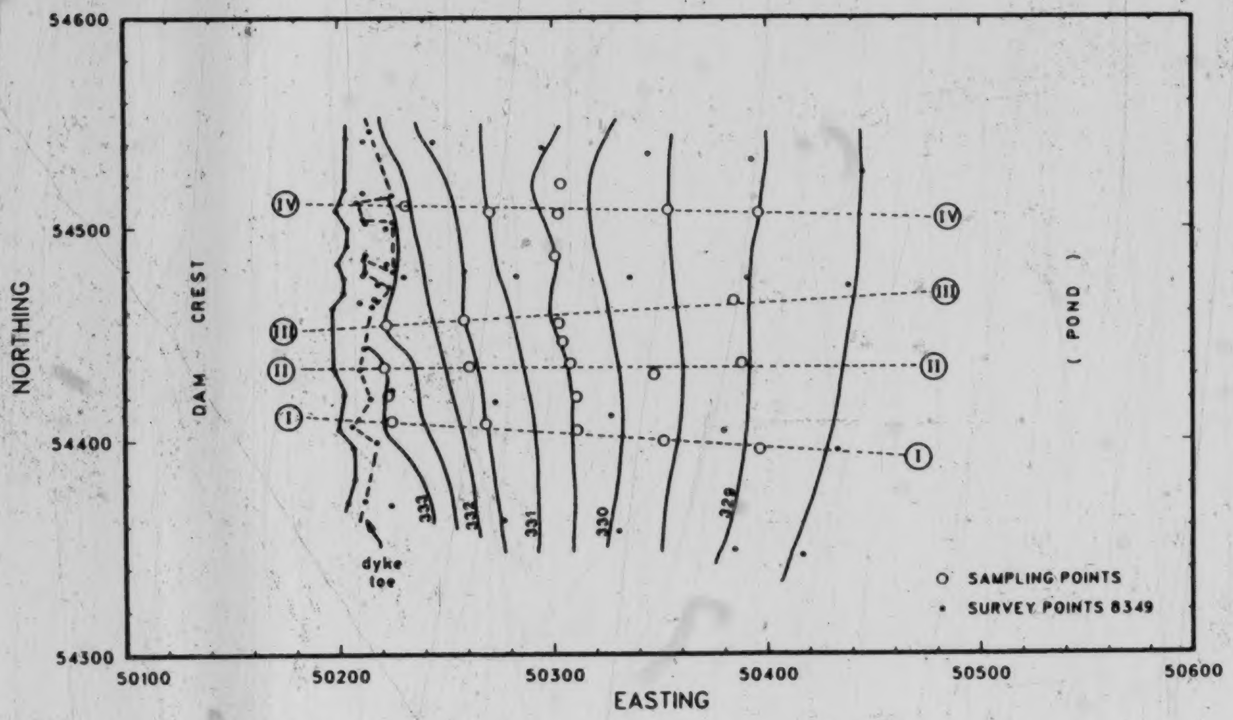

NORTHING
54600
54500
54400
54300
EASTING
50100
50200
50300
50400
50500
50600
DAM CREST
(POND)
dyke toe
I
II
III
IV
335
334
333
332
331
330
329
• SURVEY POINTS 8329

Fig. 5.38 - Contour map of the beach before Test nº 4


54600
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NORTHING
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DAM - CREST
(POND)
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III
II
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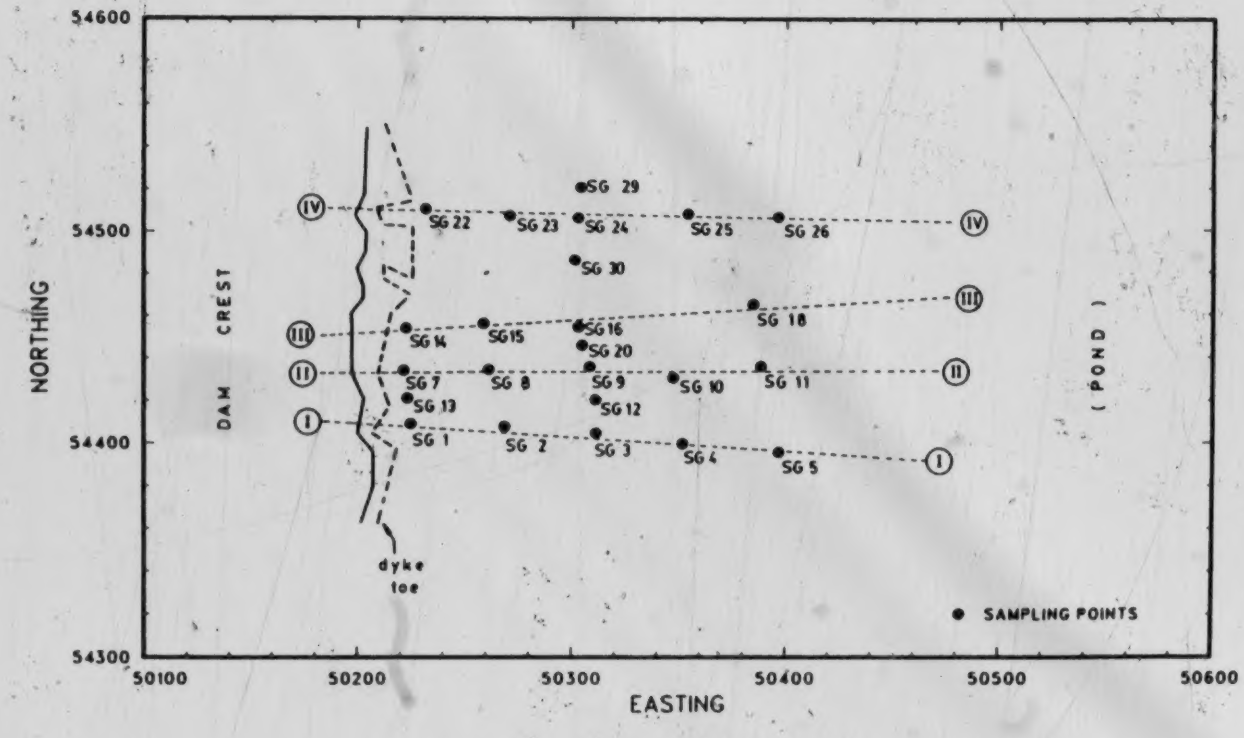
333
332
331
330
329
dyke toe
O SAMPLING POINTS
• SURVEY POINTS 8349

Fig. 5.39 - Contour map of the beach after Test nº 4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SG 23
SG 24
SG 25
SG 26
SG 30
SG 18
SG 14
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SG16
SG 20
SG 7
SG 8
SG 9
SG 10
SG 11
SG 13
SG 12
SG 1
SG 2
SG 3
SG 4
SG 5
● SAMPLING POINTS

Fig. 5.40 - Location of cross-sections and sampling points - Test nº 4

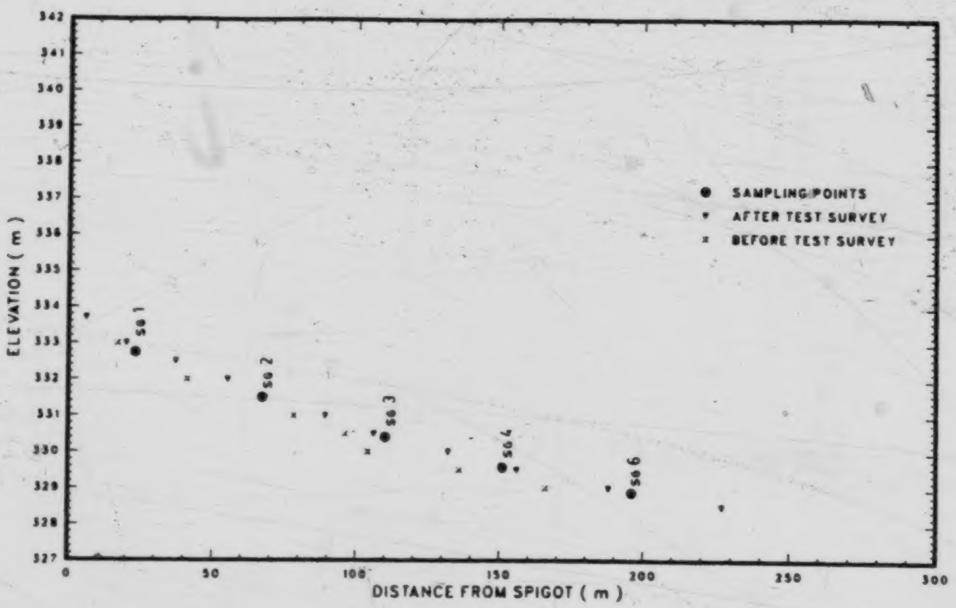


FIG. 5.41 - Profile I - Test nº 4

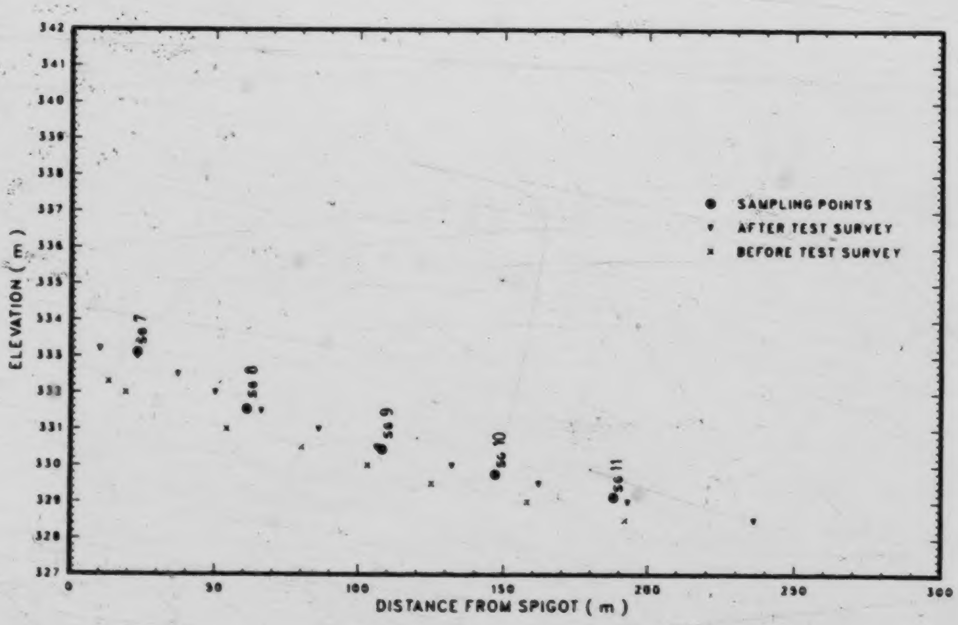


FIG. 5.42 - Profile II - Test nº 4

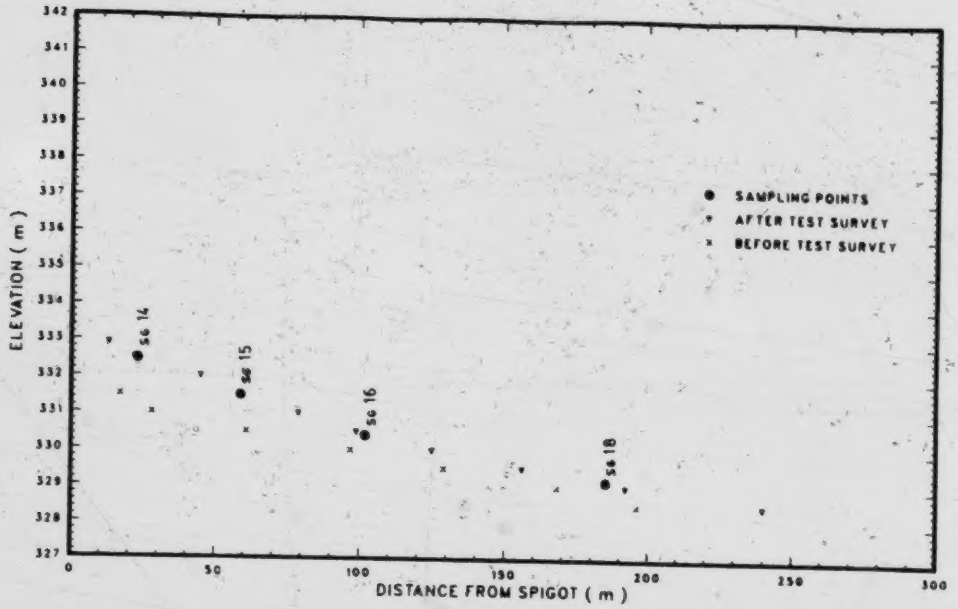


FIG. 5.43 - Profile III - Test nº 4

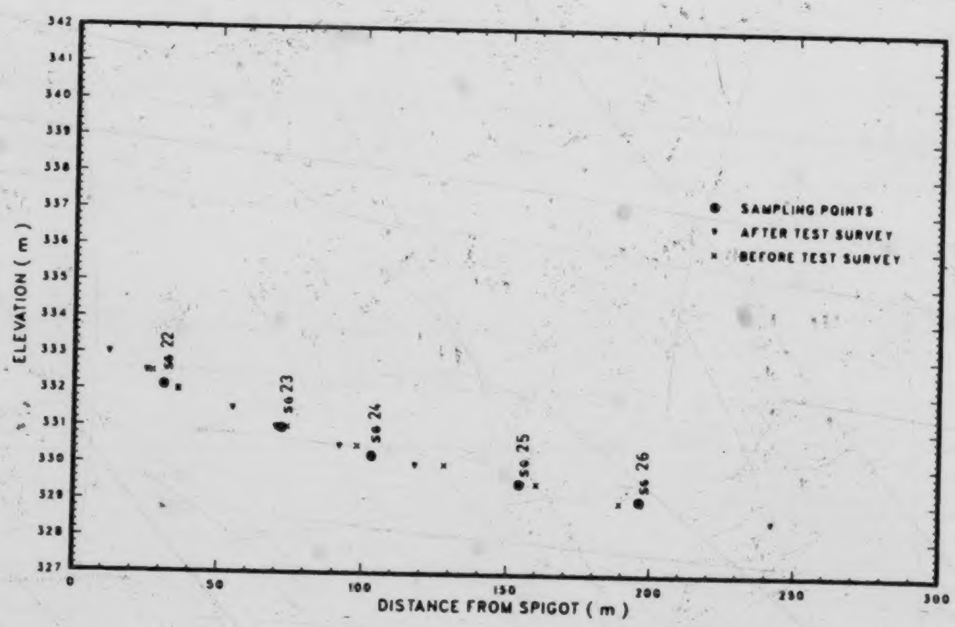
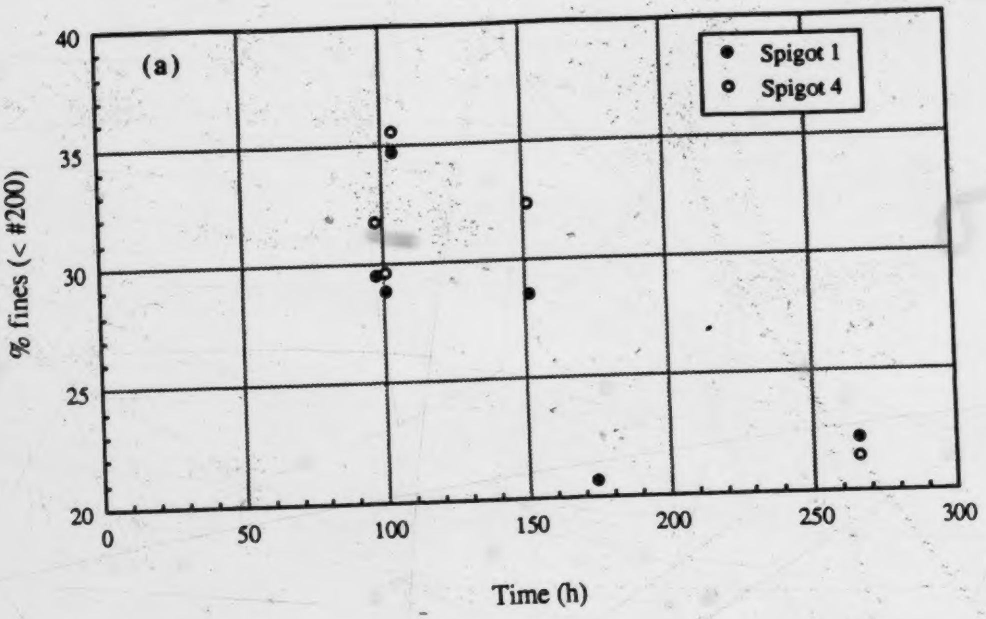


FIG. 5.44 - Profile IV - Test nº 4



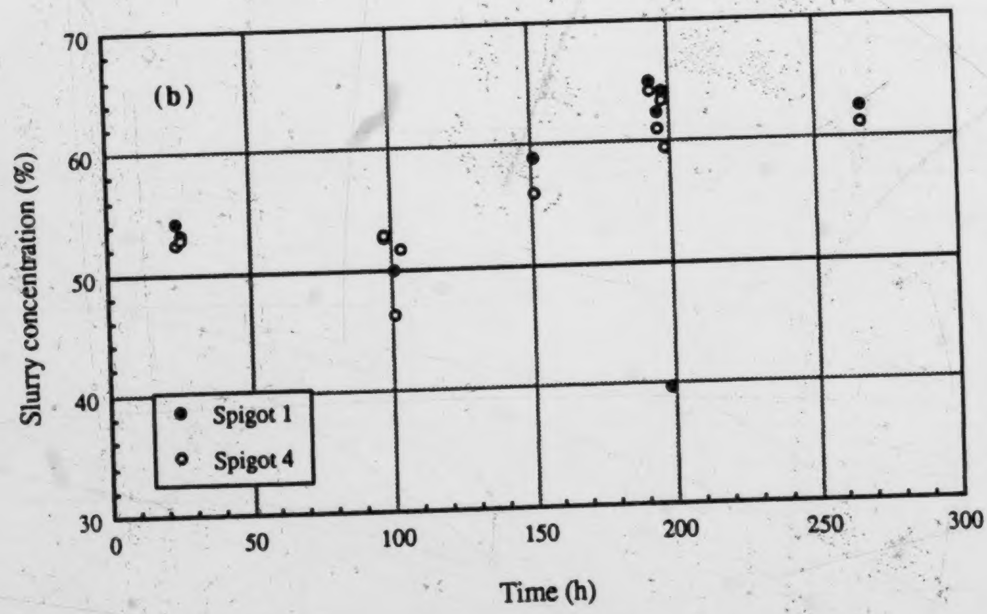
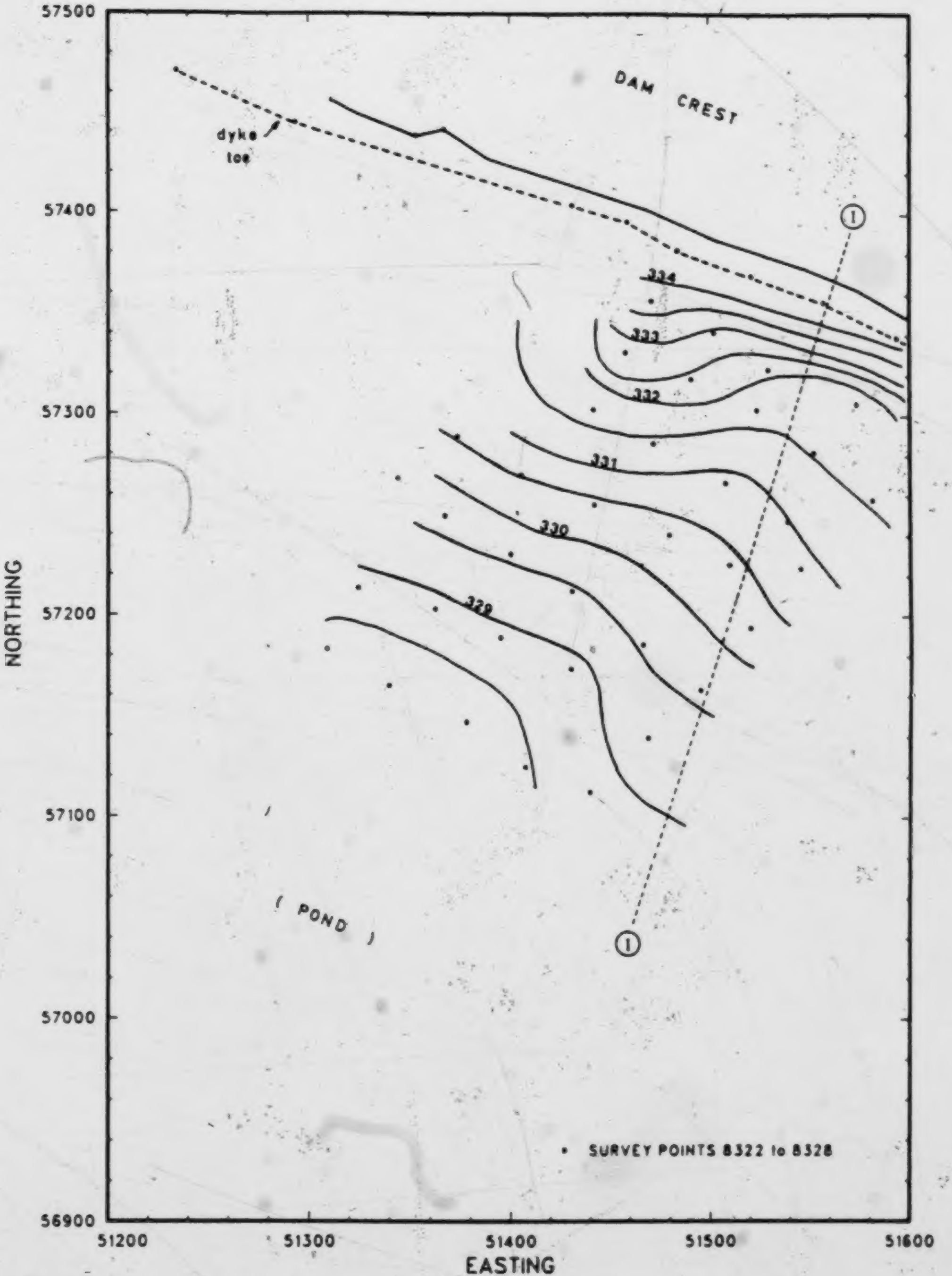


FIG. 5.45 - Variation of input parameters with time - Field Test nº 6


57500
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DAM CREST
dyke toe
334
333
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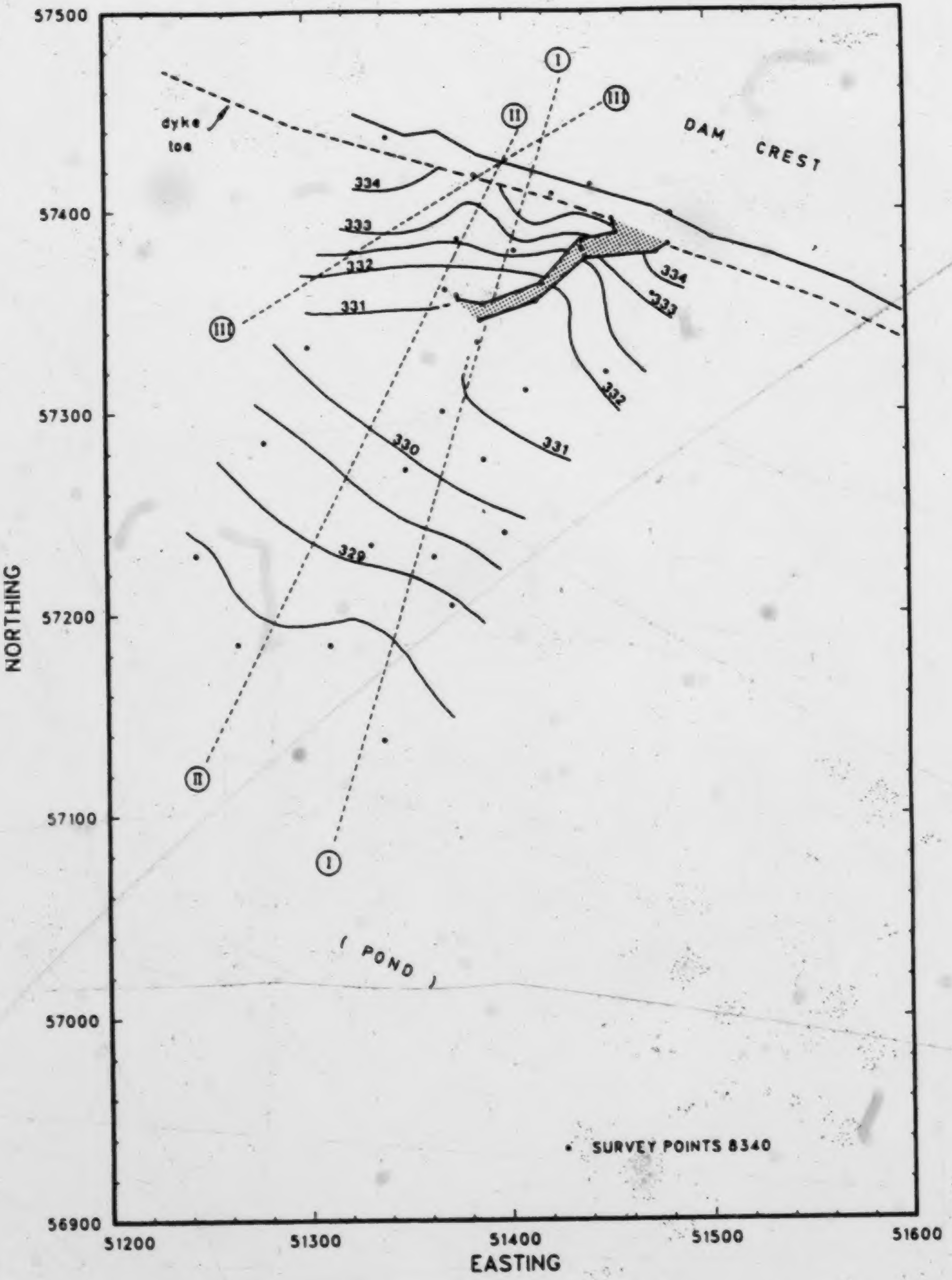
1
1
(POND)
SURVEY POINTS 8322 to 8328

Fig. 5.46 - Contour map of the beach before Test nº 6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334
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329
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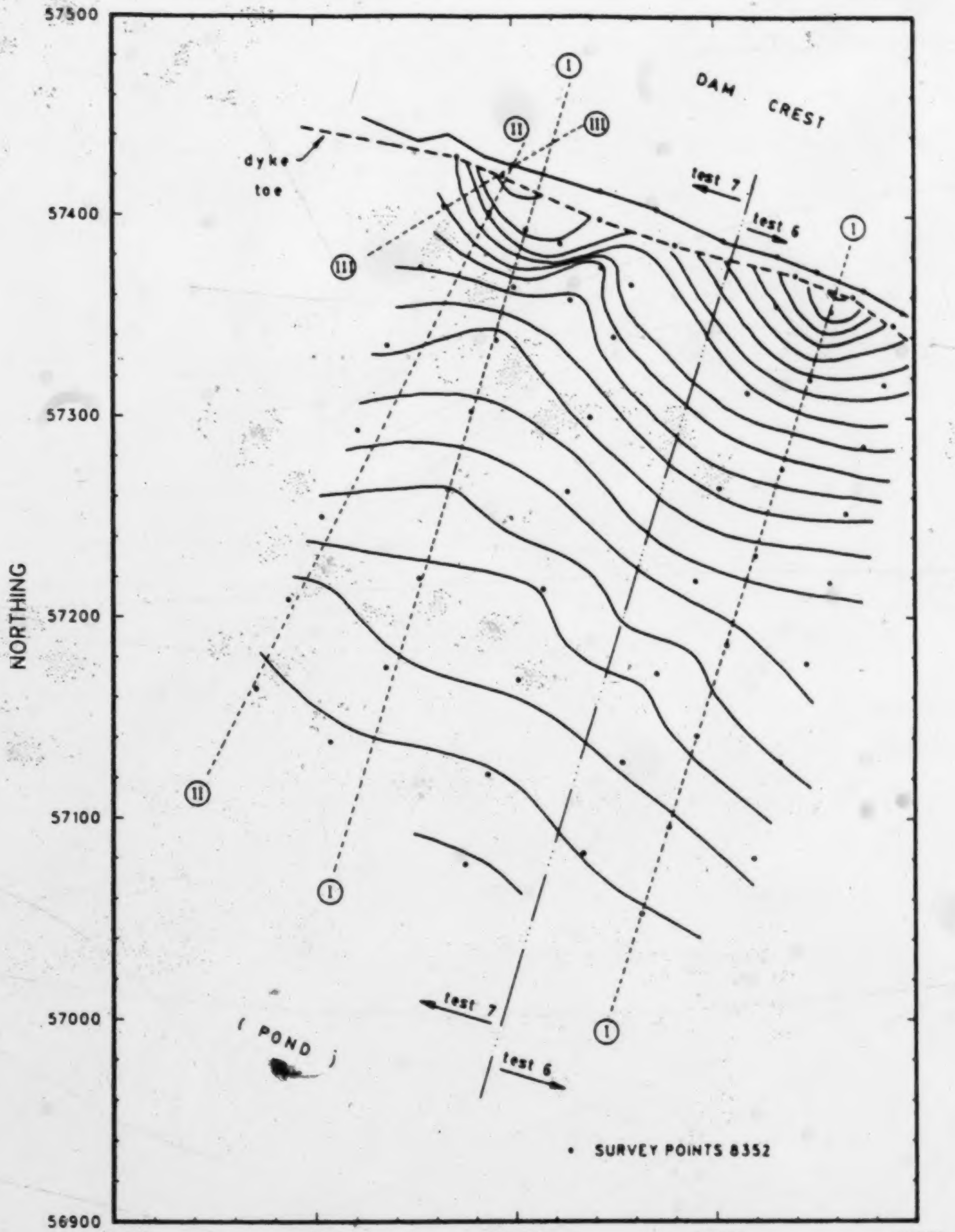
II
III
(POND)
• SURVEY POINTS 8340

Fig. 5.47 - Contour map of the beach before Test nº 7 (and after Test nº 6)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NORTHING
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51400
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EASTING
DAM CREST
dyke toe
test 7
test 6
I
II
III
(POND)
• SURVEY POINTS 8352

Fig. 5.48 - Contour map of the beach after Test nº 6 and after Test nº 7

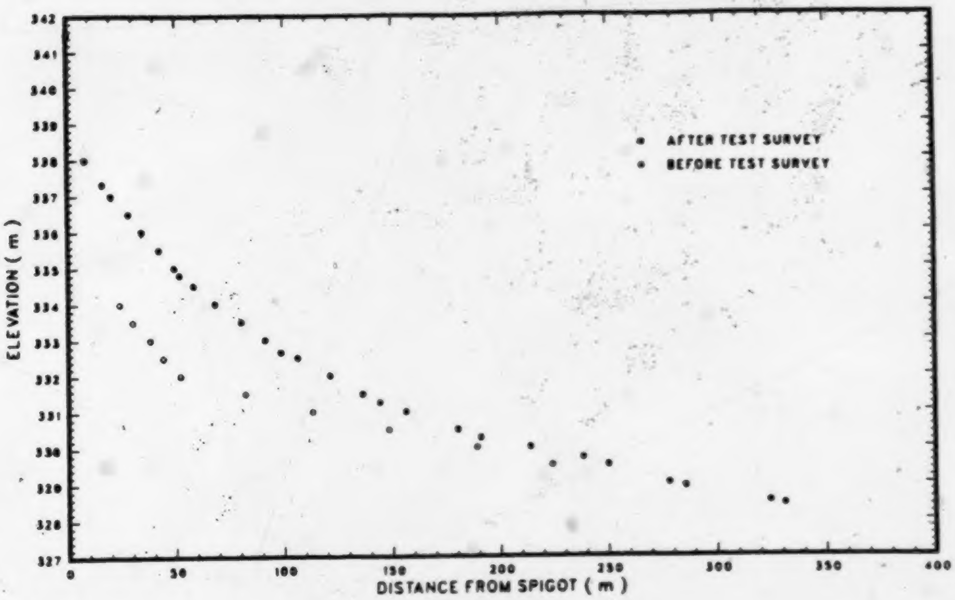

FIG. 5.49 - Profile I - Test nº 6

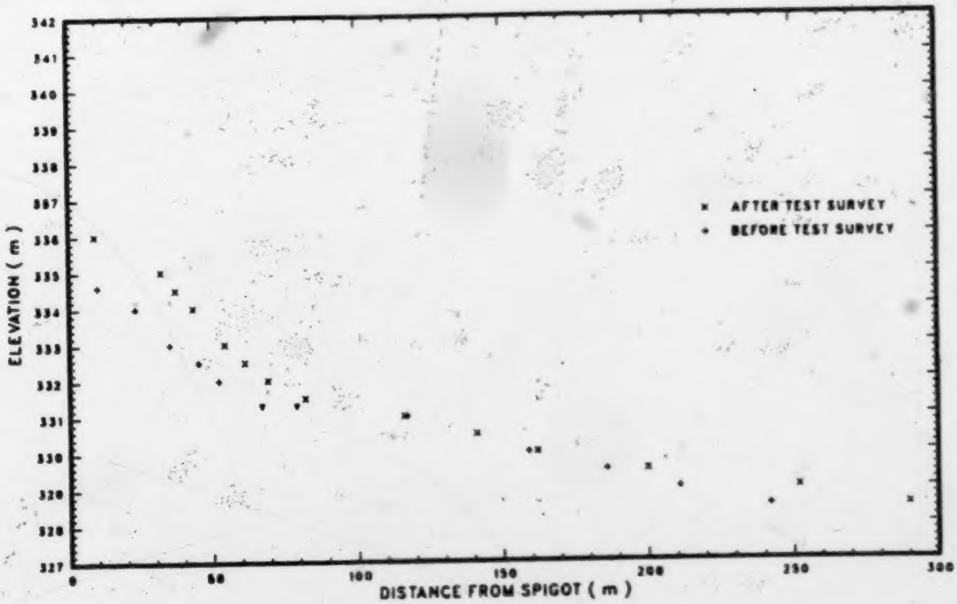

FIG. 5.50 - Profile I - Test nº 7

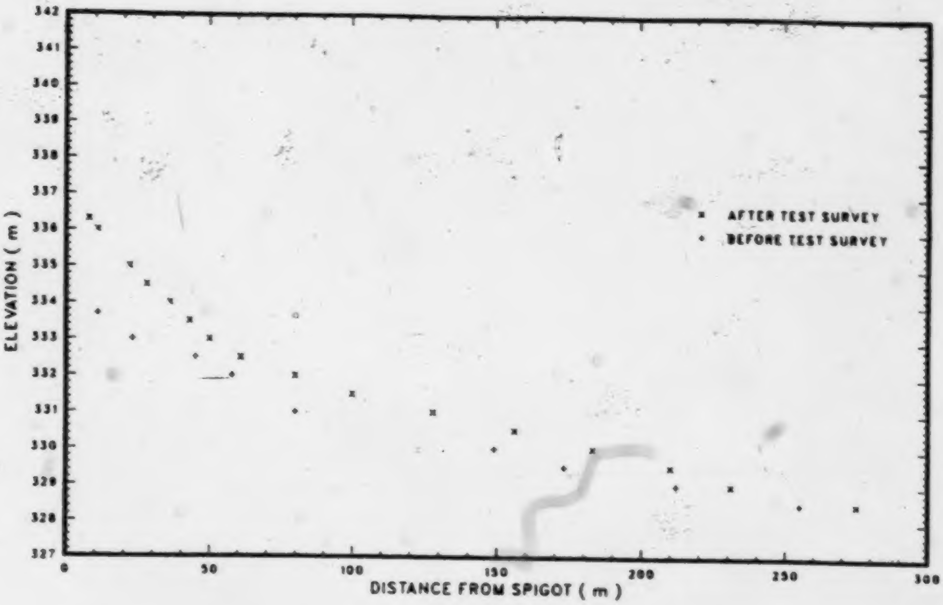

FIG. 5.51 - Profile II - Test nº 7

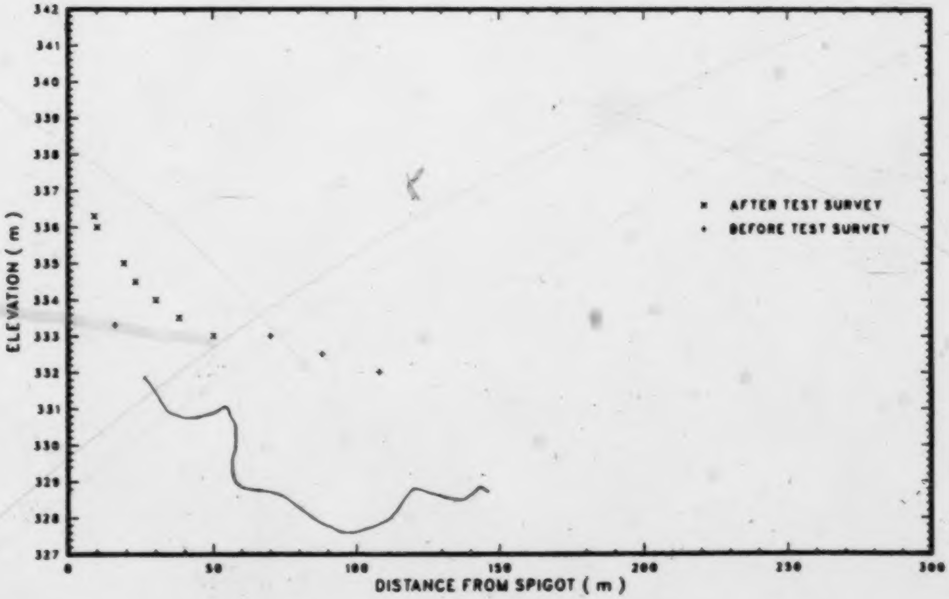

FIG. 5.52 - Profile III - Test nº 7

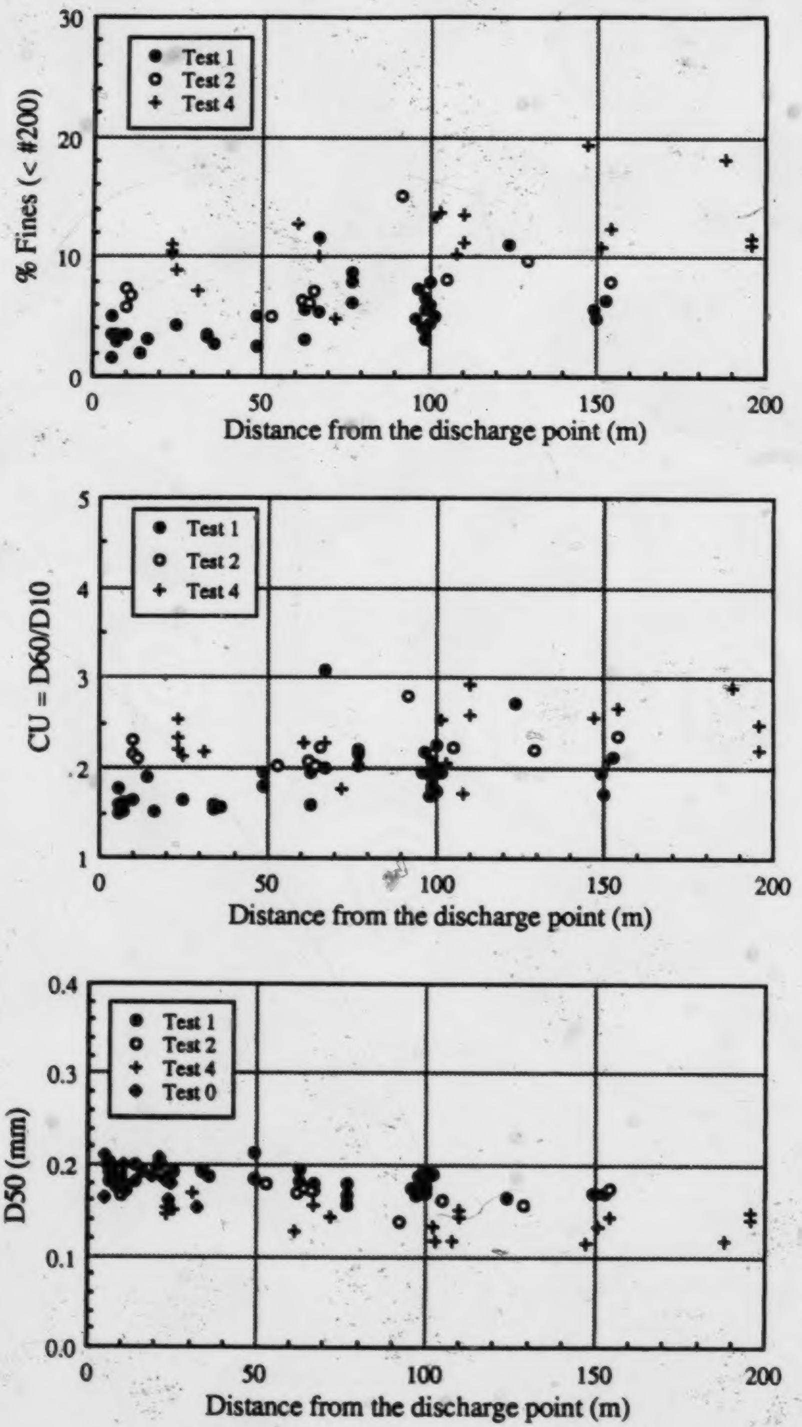

FIG. 5.53 - Variation of grain size along the beach for the various tests

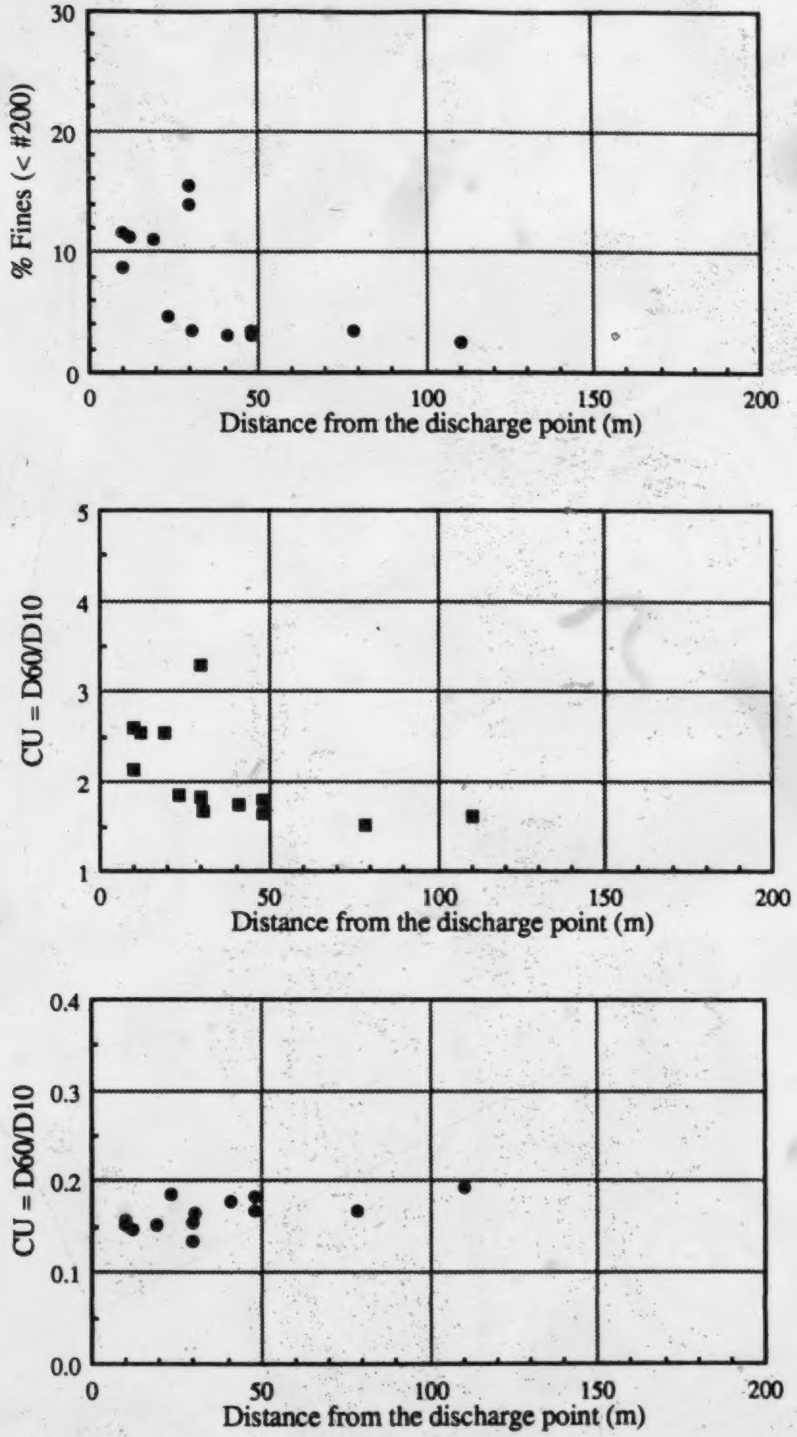

FIG. 5.54 - Variation of grain size distribution inside the deep channels (Test 2)

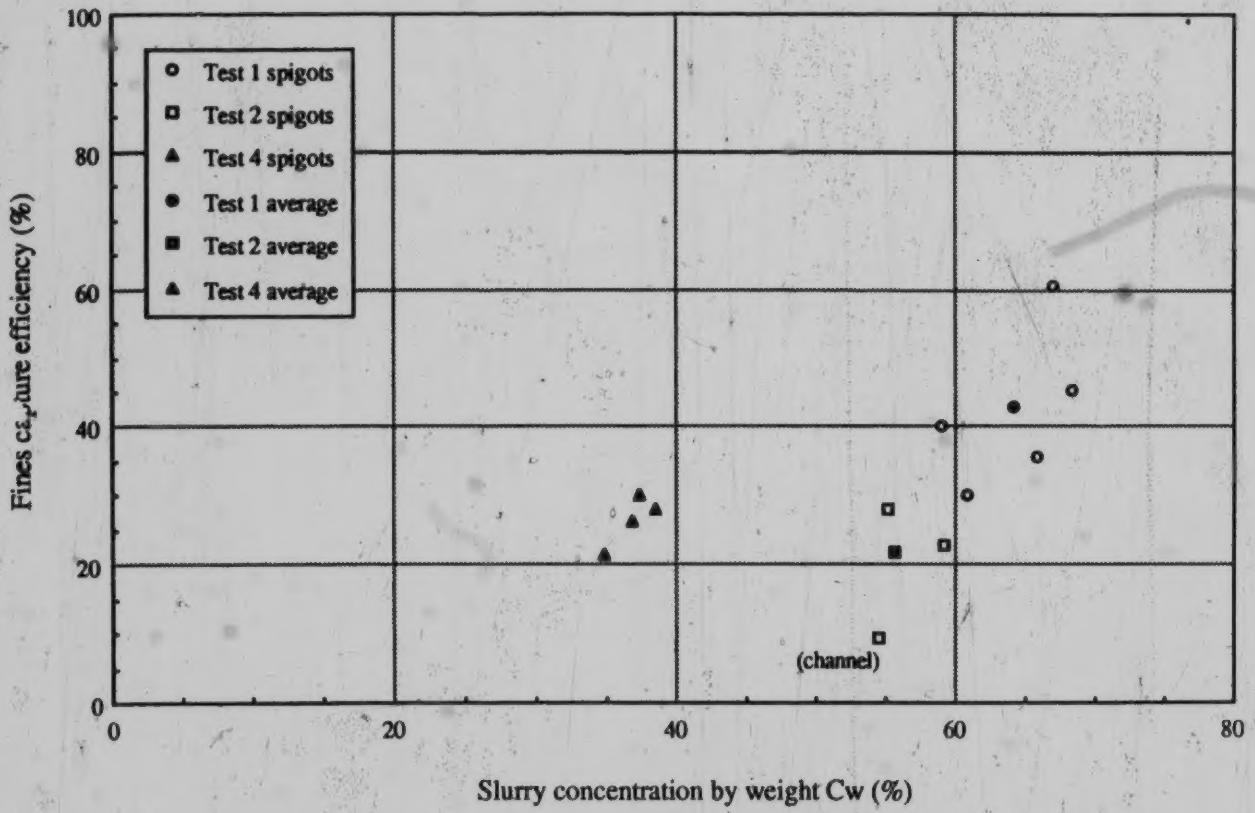

FIG. 5.55 - Effect of the slurry concentration on efficiency of fines capture

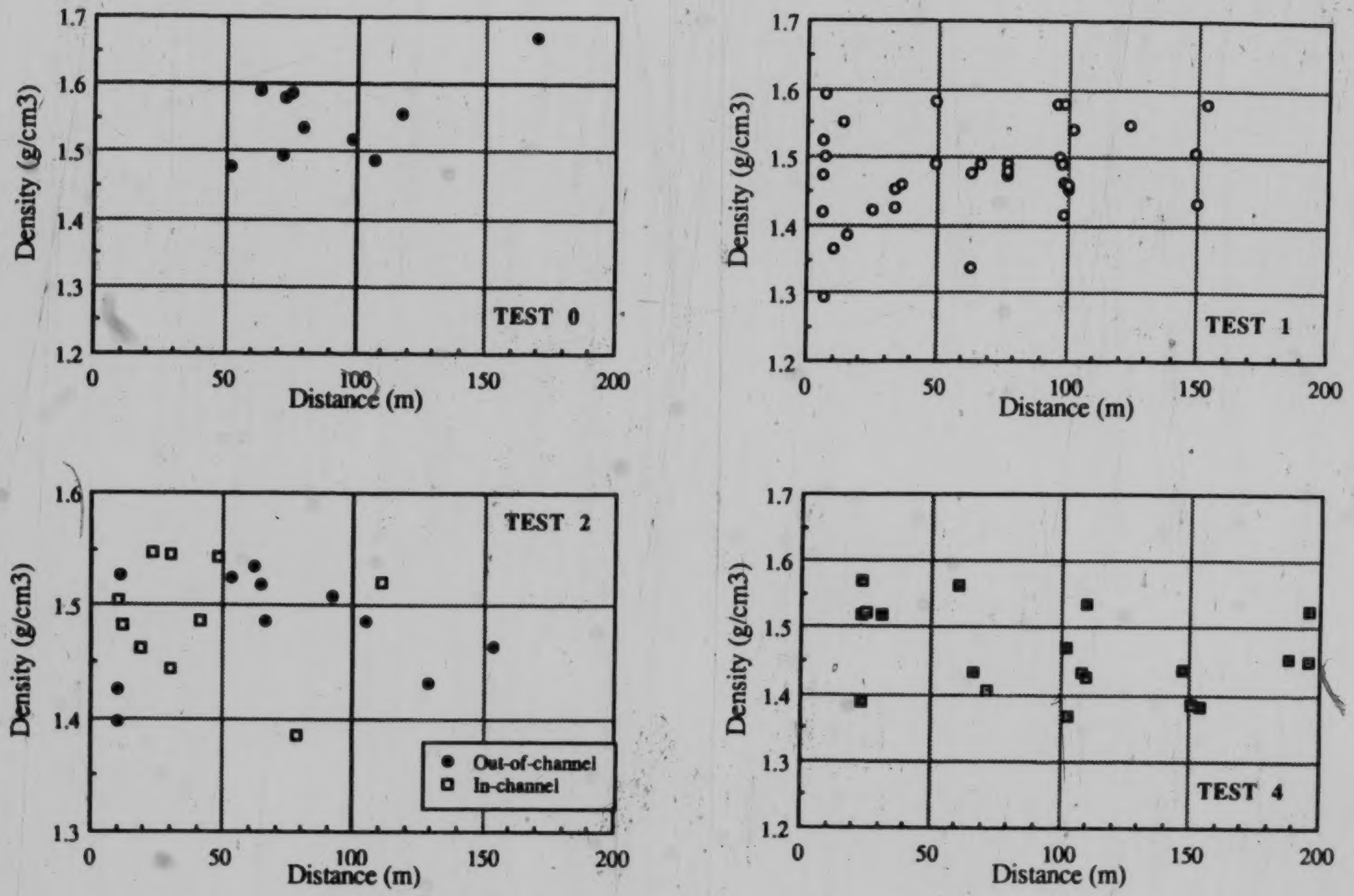

FIG. 5.56 - Variation of beach density with distance from the discharge point for the various field tests

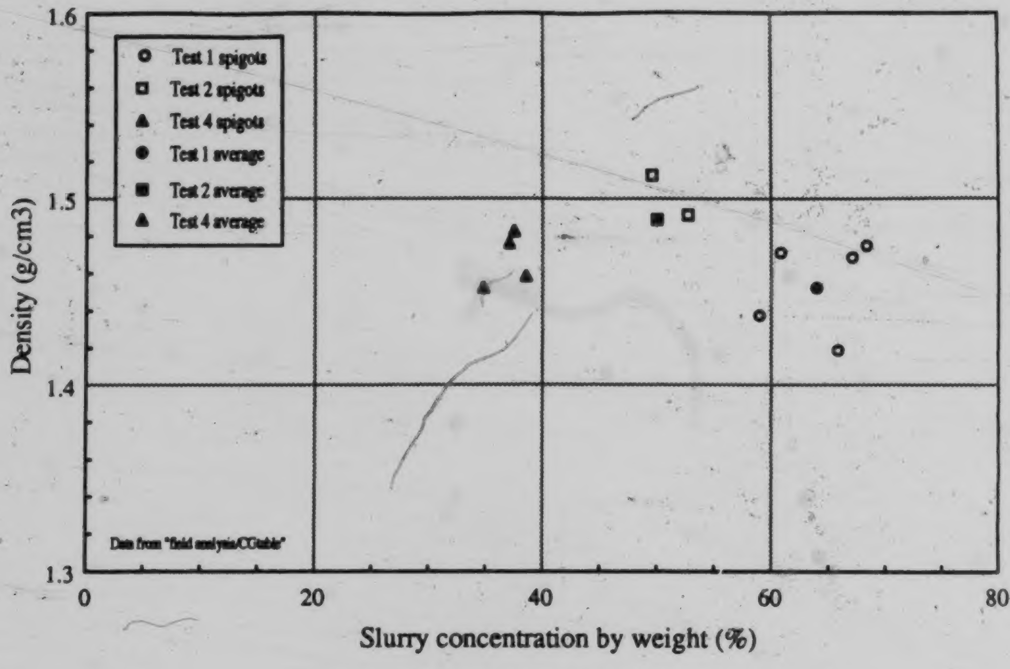

FIG. 5.57 - Effect of the concentration of the slurry on the average beach density

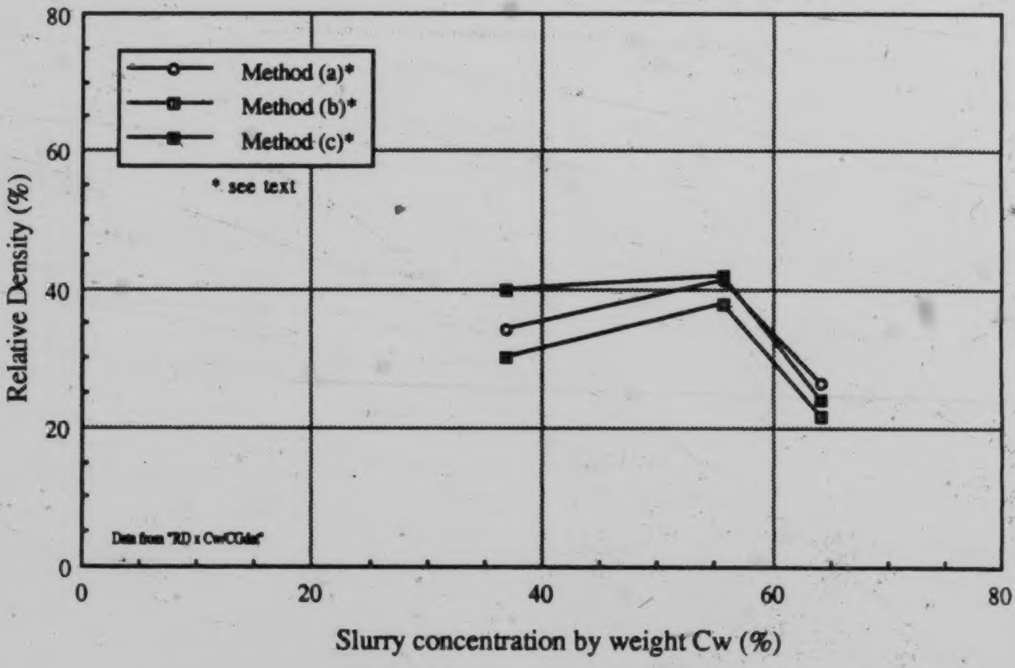

FIG. 5.58 - Effect of the slurry concentration on the beach relative density

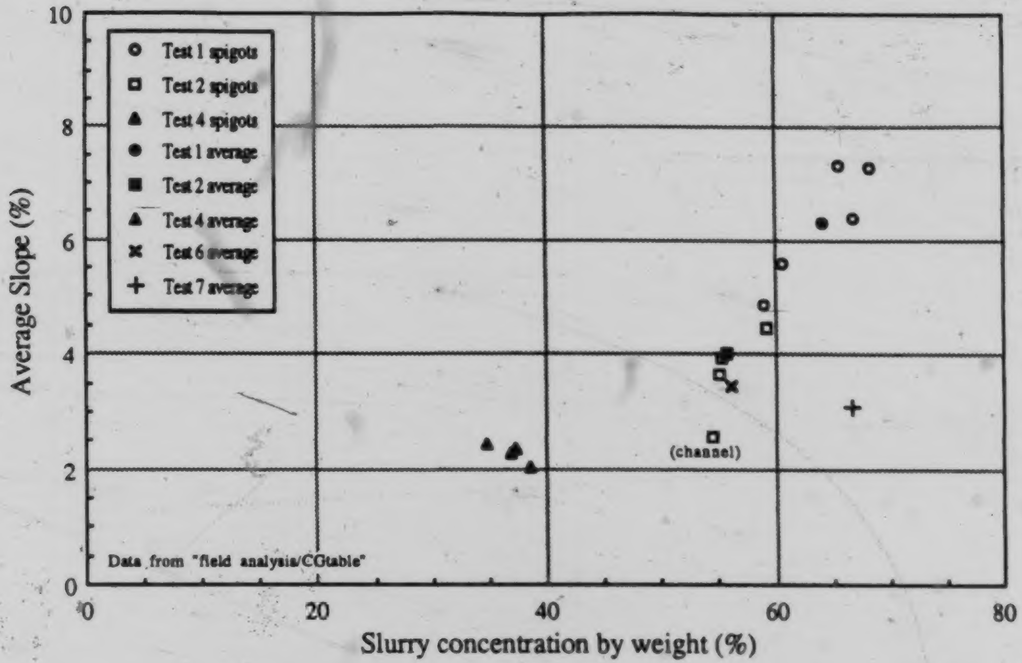

FIG. 5.59 - Effect of slurry concentration on the beach average slope for all field tests

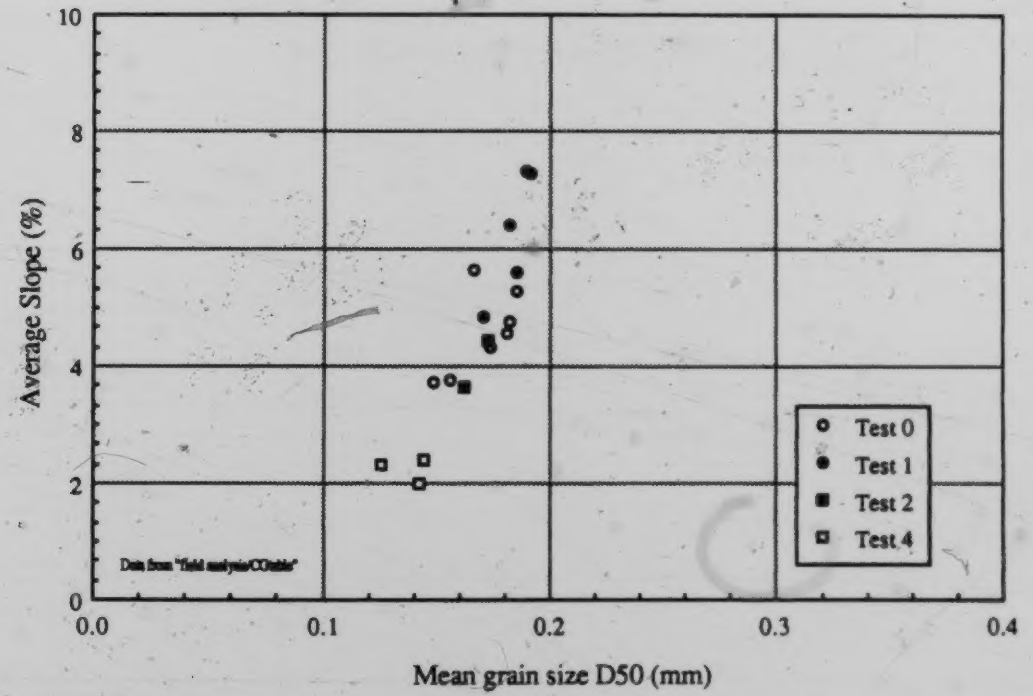

FIG. 5.60 - Effect of the mean grain diameter on the average beach slope

Chapter 6

Characteristics of Hydraulic Fills in Flume and Field Deposition Tests

6.1 - INTRODUCTION

One of the problems associated with the design of hydraulic fills is the lack of a method to estimate the properties of the fill at the design stage.

Hydraulic deposition in laboratory flumes to model field deposition is an attractive idea to study hydraulic fills. In the laboratory, it is possible to vary parameters in a controlled way to study the effects of different variables on the properties of the fill. It is also possible to simplify the conditions to permit a better understanding of the physical phenomena involved and to focus attention on aspects of the phenomena that might otherwise be overlooked or misidentified in the field. Ideally, flume tests would permit the study of the effects of variables such as slurry concentration and flow rate on the properties of hydraulic fills in order to give the optimum range for each variable to be utilized in the field and to permit the estimate of the fill properties under field conditions. However, there are several difficulties with modelling problems of sediment transport, as discussed in Chapter 3. As a result, it is still not possible to apply scaling factors to values obtained from laboratory fills to estimate the field scale characteristics of hydraulic fills, as can be done with other types of scale models.

In fact, there are no studies presented in the literature, to the author's knowledge, showing a comparison between properties of hydraulic fills from well controlled flume tests and results of field measurements in the same material in order to quantify the differences. It is known in practice though, that slopes obtained in flume experiments are much steeper than their field counterparts, but little has been said about flume values of density or grain size distribution. Hence, in this study flume and field tests were carried out using the same material in order to compare the fill parameters obtained under laboratory and field conditions (see Chapters 3 and 5). In both cases, the study dealt with subaerial hydraulic fills. The term subaerial is utilized here in the same sense as in the geology literature, i.e., a phenomenon that occurs in the open air (Webster, p.2272) as opposed to under water (subaqueous). The material utilized for the comparative tests was tailings sand from the Syncrude oil sand mine in Fort McMurray, Alberta. It is a fine uniform subangular quartz sand that has a mean grain diameter of 0.18 mm and a coefficient of uniformity D_{60}/D_{10} between 1.9 and 2.9.

This chapter compares the intrinsic characteristics of the flume and field test fills, such as grain size distribution, density and fabric, and analyzes the findings of these tests. A comparison between the geometry of the flume and the field beaches is discussed in the next chapter. However, before analyzing the experimental results, the difference between segregating and non-segregating slurries is discussed as these two types of behaviour conduce to different deposition processes.

6.2 - SEGREGATING AND NON-SEGREGATING SLURRIES

Segregation of a slurry refers to the tendency of the solid fraction (or part of it) to settle, creating a concentration gradient within the mass. When a slurry behaves as a non-segregating slurry, the solids are uniformily distributed throughout the mass. Non-segregating behaviour is common in slurries of high solids concentration and very fine particle sizes. Usually there is a sharp increase in viscosity of the slurry compared to the viscosity of the carrier fluid and the slurry displays non-Newtonian rheology. Viscous forces predominate. When segregating behaviour occurs, the solids are not evenly distributed and pronounced concentration gradients exist along any vertical axis within the mass. The fluid and the solid phases interact but retain separate identities, and the slurry viscosity remains similar to that of the carrier fluid. In this case, particle inertial effects predominate. Other terminologies found in the literature to describe this difference in behaviour include: settling/non-settling, heterogeneous/homogeneous, bleeding/non-bleeding and stream flow/mudflow. Between these two extreme types of behaviour, there is an intermediate region in which both mechanisms (viscous effects and particle inertial effects) are of approximately equal magnitude. One type of behaviour may predominate over the other one, depending on the conditions. In fact, in this intermediate region the system is often very sensitive to minor changes in its conditions (Wasp et al.,1977). Whether a slurry behaves as segregating or non-segregating depends on the:

- type of carrier fluid
- type and amount of chemicals added to the fluid
- type of solids
- grain size distribution of the solids

- proportion of solids to the total amount of slurry (slurry concentration)
- flow conditions

For a particular type of solids and fluid under constant flow conditions, the properties of the slurry depend on the relative amounts of solids and fluid, i.e., the slurry concentration. Within the solid fraction a differentiation must be made between the finer and the coarser fraction. The finer fraction of the solids controls many properties of the slurry and may have an important effect on the segregating or non-segregating behaviour. In some cases, a relatively small change in the percentage of fines may change the slurry behaviour from one type to the other. Therefore, it is convenient to treat the finer fraction of the solids separately. Utilizing these concepts, Scott and Cymerman (1984) propose a diagram to differentiate the properties of slurries using the proportion of sand, fines and water (Figure 6.1). Boundaries of distinct behaviour (segregating / non-segregating, pumpable / non-pumpable, solid / liquid, etc.) can be located on this diagram, that then becomes a convenient tool to analyze the behaviour of a particular type of slurry, given the sand, water and fines content. Scott and Cymerman (1984) also discuss the definition of what constitutes the finer fraction, as it may take different values for different cases.

The behaviour boundaries presented in Figure 6.1 correspond to Syncrude's tailings. The boundary segregating/non-segregating slurry was determined under static conditions (slurry placed in a 90 cm high standpipe) and as such, it is not valid for hydraulic deposition. This boundary is different for static and dynamic conditions because a slurry that segregates in a standpipe may behave as non-segregating under dynamic flow conditions. Also, segregating/non-segregating behaviour depends on the flow conditions themselves. Slurries flowing at a lower energy level (such as in flume tests) display a different behaviour compared with the same slurry flowing at higher energy levels (such as under field conditions).

For flow of slurries in pipelines, there are several empirical criteria to define the slurry behaviour (see Vanoni, 1975 and Wasp et al.,1977), however similar criteria have not been developed for hydraulic fills. An additional factor to be considered for hydraulic deposition is drainage. A slurry flowing on a pervious surface such as the deposition beach may loose water through seepage and become a non-segregating slurry. The opposite may occur towards the end of the beach, where seepage flow may emerge from the slope. This change in slurry behaviour along the depositional surface has been reported for alluvial fans. It has also been observed in the flume tests described in Chapter 3.

Whether the slurry behaves as segregating or non-segregating during deposition has a major impact on several aspects of the resulting deposit, such as geometry, density, grain size distribution, fabric, etc. The existence of both types of slurry behaviour has been recognized both in the hydraulics/sedimentology literature (mudflow or debris flow versus stream flow) and in the mining literature (where it is sometimes referred to as bleeding/non-bleeding slurries) The difference between the deposits formed by each type of slurry has been extensively studied (Blissenbach, 1954; Hooke, 1967, 1968; Bull, 1968, 1972; Beaty, 1970; Lustig, 1974; Macke, 1977; Weave , 1984; Pierson and Scott, 1985 among many others).

There is a significant difference in geometry between deposits formed by segregating or by non-segregating slurries. Segregating slurries produce concave slopes both in the field and in the laboratory. However, working with non-segregating slurries, Fourie (1988) obtained convex profiles in a laboratory flume. Also deposition of non-segregating slurries produces steeper slopes than deposition of segregating slurries of the same material (Robinsky, 1978).

Segregation also has a direct effect on the grain size distribution in the deposit. Flow of segregating slurries causes hydraulic sorting of particles, with different size fractions being deposited at different locations along the flow path. Deposits formed by non-segregating slurries do not develop hydraulic sorting, therefore the coefficient of uniformity is relatively high and the grain size distribution remains approximately constant along the deposit. Clearly, fines capture (see Chapter 5) and hydraulic sorting are two different expressions of the same phenomenon and fines capture should decrease as hydraulic sorting becomes more accentuated and increase when sorting attenuates. In the extreme case of non-segregating flows, there is no hydraulic sorting and consequently fines capture is maximum (*FC* = 100%). Accordingly, for the field deposition tests described in Chapter 5, the fines capture efficiency increased for higher slurry concentrations.

In segregating slurries, the fluid and the solid fraction behave independently, so the fluid is able to apply hydrodynamic forces on individual particles, especially if the solids concentration is low. This contributes to hydraulic sorting and to the organization of particles in the deposit, forming a characteristic stratigraphy (which also depends on the flow conditions). Consequently, the flow of segregating slurries deposits a relatively dense material with a pronounced grain arrangement. In contrast, deposits formed by non-segregating slurries are massive, with a lack of stratigraphic features and with no preferred grain orientation. The high rates of deposition associated with non-segregating slurries result in a lower density than the material would have if deposited under segregating flow conditions (Kolbuzewski, 1950; Allen, 1982, 1985).

Therefore, the distinction between segregating and non-segregating behaviours is essential for the understanding and analysis of the deposition process as well as the resulting hydraulic fill.

As the majority of the hydraulic fills are built using segregating slurries, the remainder of this chapter will discuss particularities of deposition from this type of slurry. Due to the marked difference in behaviour between segregating and non-segregating slurries it is important to note that all the statements below are only valid for hydraulic deposition of segregating slurries.

6.3 - FLOW CHARACTERISTICS ON HYDRAULIC FILL BEACHES

Since hydraulic fills are formed by a sediment laden flow that deposits solids as it moves downstream and different flow conditions result in different deposits, the importance of studying the characteristics of this flow is obvious. The knowledge of the flow conditions on the beach makes it possible to infer what kind of sedimentary deposit is being formed, which in turn will provide information on the fill characteristics that can be expected and the kind of heterogeneities that can occur.

6.3.1 - Normal flow conditions

The typical flow of a segregating slurry on a hydraulic fill beach is a complex combination of sheet flow, braided flow and meandering channels with associated formation of bars, islands, and lag deposits.

Sheet flow commonly occurs right after the discharge point, where the flow is spreading but the specific flow rate is still relatively high. After the flow has opened up significantly and some fluid has been lost through seepage, the flow compensates the smaller specific flow rate by braiding. Islands and bars are formed, so the actual flow width is reduced. Further downstream, as seepage water flows out of the slope, the degree of braiding decreases and eventually the flow may return to sheet flow. This drainage pattern of sheet flow close to the discharge point followed by braided flow seems to be the most common in hydraulic fills (Vick, 1983; Rice, 1989; Chapter 5), however situations of braided flow preceding and transforming into sheet flow down the beach have also been reported (Conlin, 1985 and French, 1987). In some cases, instead of spreading out after the discharge point, the flow concentrates in channels that are usually relatively shallow and temporary. Deep irreversible channels may also occur and are discussed in the next section.

In any case, a continuous variation in type and strength of flow in time and space prevails, causing the deposit characteristics to vary frompoint to point. Typically, a sandy hydraulic fill presents heterogeneities on the scale of centimeters to meters in plan and on the scale of millimeters to centimeters in depth.

The most common bedforms observed during the deposition tests described in Chapter 5 were upper-stage plane beds, antidunes and chutes-and-pools. Upper-stage plane beds were usually found in divergent flows close to the discharge point, in localized areas of the lower part of the beach and on secondary flows. High energy antidunes were widespread over most of the upper beach. Flow concentrations in this area formed chutes-and-pools, which moved slowly upstream remolding the beach material. On the lower part of the beach, antidunes commonly occurred as trains of standing waves. Although not using this terminology, Bentel (1981) also described upper-stage plane beds and chutes-and-pools on a platinum tailings beach.

The flume tests described in Chapter 3 seem to have reproduced adequately the flow conditions observed in the field (see Chapter 5). The flow in the flume formed sheet flow, braided flow and meandering channels with antidunes and upper stage plane beds in a manner similar to the field flow. However, hydraulic jumps and chutes-and-pools were more localized and less common in the flume than in the field, which could be explained by the lower energy level of the laboratory flow.

The flow features described above - drainage pattern and bedforms - define the deposition process, and therefore affect the fill properties. Other details such as jet angle, height between the beach and the spigots, formation of a plunge pool, existence of back-water, rate of filling, interaction between the subaqueous and the subaerial environments, etc. seem also to be important for the properties of the fill and should be monitored.

6.3.2 - Deep channelling condition

This condition refers to the formation of deep channels on the beach, which convey all the flow directly to the pond, ceasing beach deposition (see Chapter 5). For the case of Syncrude's tailings dyke described in Chapter 5, these deep channels were 1 to 12 meters deep and capture the total flow of one or more spigots. All the solids are discharged to the pond, causing a noticeable increase in pond water turbidity in front of the channels. Although temporary cycles of deposition in the channels may occur, these channels may persist for weeks without much change to the bottom elevation.

This deep channelling phenomenon can have serious consequences to the operation of decant systems, to the maintenance of adequate free board and to the lifetime of waste disposal facilities and can completely impair the construction of hydraulic fills. Moreover, this condition causes a substantial increase in the amount of material deposited under water and therefore, in a very loose and liquefiable state.

A similar phenomenon also occurs on alluvial fans and it is called fan head entrenchment in the alluvial fan literature. The so-called fan head trenches are channels that develop at the apex region of the fan and convey most of the sediment to a down fan area. Typically these trenches range in depth from a few meters to tens of meters. When the alluvial fan progrades into a water body (in which case the alluvial fan is called fan delta), the fan head trench or channels usually transports all the sediment to the water as in hydraulic fills. Fan head trenches are described in detail in the alluvial fan literature because it is one of the most important features that govern the nature and distribution of sediments, as they are in hydraulic fills when they occur.

The bottom of the deep channels (or trenches) is always flatter than the surface of the beach or fan, and therefore the channel intersects the fan surface at a point. In hydraulic fills and fan deltas, this intersection may be above or below the water level.

After the intersection point, the flow spreads out and becomes a sheet flow or a braided network of shallow channels, what reduces its sediment transport capacity and causes deposition. This region becomes the major locus of deposition. Coarse material tends to deposit immediately downstream of the intersection point.

Nick points that progressively move upstream were observed in several instances during the field tests (Chapter 5). Shortage of sediments causes the appearance along the channels of nick points (Weaver, 1984) and according to Harvey (1980) an increase in sediment supply results in elimination of the nick points. Nick points can also be caused by a variation of the water level downstream, underwater slope instability or an overall movement of the beach. When a nick point is moving upstream in a channel that bifurcates, it moves up both branches (Jackson, 1981; Weaver, 1984). In laboratory experiments on fan deltas, Jackson (1981) observed that the presence of single or multiple nick points in channels is part of the normal mechanism of slope adjustment. The relief of nick points tends to decrease as they move upstream. Begin (1978) modelled the existence and behaviour of nick points using relatively simple equations that are based on heat transfer equations. For hydraulic fill monitoring, nick points provide an important indication of an adjustment in slope probably caused by a change in the deposition boundary conditions.

Since the first studies on alluvial fans late last century, many authors have described fanhead trenches and discussed their causes, characteristics and consequences. Schumm et al.(1987, p.283) summarized 16 causes of fanhead entrenchment as identified by several authors (see Table 6.1). Based on the experimental results described in Chapters 3, 4 and 5, it is possible to classify these causes into three categories:

1) increase in flow rate: causes 1?, 2, 4?, 5, 8, 9, and 10;
2) decrease in solids concentration of the feed: causes 3, 7, 11 and 16; and

3) artificial oversteepening of the original equilibrium slope: causes 6, 12, 13, 14 and 15.

This shows that the actual cause of fan head entrenchment (and deep channel formation) is that the slope became too steep for the incoming flow conditions either because the slope was artificailly oversteepened or because the incoming flow varied. Both situations were then investigated to determine the cause of the channelling problem observed during the field tests.

Artificial oversteepening of the slope could have been caused by movement of the whole structure or by lowering of the pond water level. The first alternative was ruled out by the inclinometer readings. The pond water level records did not show any considerable variation, however the readings were not taken frequently enough to eliminate the possibility of a sudden variation of relatively short duration. Under water slope instability can also trigger deep channelling by oversteepening the original slope.

The other class of causes of deep channel formation was variation of the characteristics of the incoming flow. The concentration of the slurry being discharged throught the spigots varied with time as shown in Chapter 5, however points of low concentration did not coincide with observations of initiation or re-establishment of channel erosion. Weaver (1984) also noticed that minor fluctuations of sediment load did not correlate well with periods of fan head aggradation or incision. These observations could possibly be explained by the frequency and accuracy of the measurements and by the time lag characteristic of geomorphic processes. Also Weaver (1984) observed that the time required to backfill the trench until the flow can spread again over the fan surface is much longer than the time require to incise the same trench. Therefore a good time correlation between feed variations and incision or aggradation phenomena possibly might not be expected.

Another point to be examined is the fact that once the incision process starts, it "feeds" itself. The confinement of the flow in a channel causes an increase in velocity that is associated with an increase in specific flow rate. Due to the increased velocity and specific flow rate, the erosive power of the flow is increased, causing further incision of the channel. Once a channel is incised and it is deep enough to contain the flow, the flow cannot spread out and deposit its load due to the imposed reduction in velocity and specific flow rate. Consequently, even if the subsequent flow in the channel has a higher concentration, the confined flow will not deposit slopes as steep as it would if it was on the beach surface. Such a channel could only be filled by a flow with substantially higher sediment concentration and lower flow rate and with a duration that is long enough to cause deposition and overflow.

Severe channelling such as described above for the field conditions had not being observed in the laboratory under normal deposition conditions. Since two of the hypotheses considered for the causes of deep channelling in the field (sudden lowering of the pond level and sudden drop in slurry concentration) could be easily reproduced by the laboratory apparatus, they were investigated. A sudden drop in the baselevel water in the laboratory caused the formation of two large channels that moved progressively upstream but did not reach the discharge point. A decrease in slurry concentration or increase in flow rate during deposition promptly caused the formation of "deep" channels in the deposit being formed in the flume. These experiments showed that the flume tests were successful in reproducing the field situation and that if channelling had not being observed in the flume, it was because the input parameters were kept constant while they were somewhat variable in the field. These tests also emphasize the power of laboratory experiments in isolating variables and testing hypothesis.

This discussion emphasizes the importance of having constant input parameters during the construction of hydraulic fills to ensure an efficient building process.

6.4 - GRAIN SIZE DISTRIBUTION ON HYDRAULIC FILLS

A sediment laden flow of segregating behaviour tends to deposit grains of different sizes in different locations. In general, the sediment transport capacity of the flow decreases progressively and as a result larger grains tend to be deposited first and smaller grains are deposited further downstream. Consequently, the typical grain size distribution along a hydraulic fill is characterized by a decrease in mean grain size (D_{50}) and an increase in the amount of fines ($\%F$) with distance from the discharge point.

This separation of hydraulically deposited materials in different granulometric fractions due to the hydraulic characteristics of the flow is called hydraulic sorting. This phenomenon has also been called hydraulic segregation (Morgenstern and Küpper, 1988; Fourie, 1988; Lighthall et al., 1989; Conlin, 1989; among others), however the term hydraulic sorting is preferred to avoid confusion with the distinct phenomenon of slurry segregation discussed in Section 6.2.

According to the Hydroprojekt Seminar (1973), hydraulic sorting starts being observed at $C_U = D_{60}/D_{10} = 1.3$ to 1.6, but up to $C_U = 2$ the fill can be considered homogeneous. The Soviet standard specification on hydraulic fills (SNIP-II-53-73) states that hydraulic sorting must be considered in all cases where

$$D_{90} / D_{10} \geq 5 \qquad (6.1)$$

and/or

$$D_{60} / D_{10} > 2.5 \qquad (6.2)$$

The sands used in the flume and field tests had values of D_{60}/D_{10} and D_{90}/D_{10} just above these limits and displayed hydraulic sorting.

The field tests showed a general tendency of decrease in mean grain size with distance from the discharge point (Figure 6.2). This trend is consistent with observations from other hydraulic fills found in the literature (Volnin and Pinalov, 1976; Soderberg and Bush, 1977; Volpe, 1979; Blight and Steffen, 1979; Vick, 1983; Havard, 1987). However, inside deep channels, the mean grain size tends to increase with distance along the flow.

The flume tests described in Chapter 3 also showed an increase in grain size with distance from the discharge point, similar to the deep channels in the field. The patterns of variation of mean grain size with distance along the flow for the field tests and for the flume tests are compared in Figure 6.3. To permit such comparison, the distance from the discharge point was normalized dividing by the total length of the beach in each case. In this figure it is clear that, although scattered, the values of D_{50} in the flume are in the lower range of the field values close to the discharge point but become larger than the field D_{50} for the second half of the beach. The variation of mean grain size inside the deep field channels follow the same trend as observed in the flume tests. Therefore the flume tests described in Chapter 3 were adequate, as designed, to simulate the granulometric distribution in the deep channels but not on the beach surface.

Observations of grain size in rivers and in flume tests carried out for hydraulic studies have shown a decrease in mean grain size with distance along the flow. However there are a few exceptions, such as some of the flume tests reported by Brook (1958), Kennedy (1960) and Guy et al.(1966). In these cases, when the sediment transport rate was very high, the bedload material was coarser than the bed material, which means that the smaller grains were left on the bed while the larger particles were being transported. Consequently an aggrading bed under this flow condition will have a mean grain size increasing with distance along the flow. A similar trend has also been observed in natural and experimental alluvial fans (Hooke, 1967). While on the surface of the fan the mean grain size had a tendency to decrease with distance, it increased along the bottom of the channels.

This phenomenon has not been studied in detail and its causes are not well understood. It seems that, although larger grains tend to deposit sooner than smaller grains when they are in suspension or waterborne in a saltating trajectory, when the dominant transport mechanism is rolling or sliding, larger grains are likely to move farther because they have a larger area exposed to the flow and less chances of imbricating than smaller grains. In any case, the experimental evidence indicates that the pattern of increasing mean grain size along the flow seems to be associated with high sediment transport rates.

Hydraulic sorting tends to be more pronounced for higher flow rates, lower slurry concentrations and relatively small flow velocity on the beach (Yufin, 1965; Melent'ev et al., 1973). This observation is consistent with the above discussion because these parameters will enhance the segregating characteristics of the slurry and minimize hindered effects while keeping the sediment transport rate relatively low.

6.5 - DENSITY OF HYDRAULIC FILLS

Laboratory flume studies of density of hydraulically deposited sands showed a trend of densities decreasing as the slurry concentration increased and increasing as flow rates also increased (see Chapter 3). Also comparing three subangular quartz sands containing almost no fines, it was found that under the same conditions the density increases for larger mean grain sizes, as expected.

The field results described in Chapter 5 were not so clear in relation to the trends of variation of density, partially because several variables changed from test to test, i.e. slurry concentration, slurry composition and height from spigots to beach, making it difficult to isolate the effects of each one. The densities obtained in the field beaches are plotted in Figures 6.4 and 6.5 as a function of the mean grain size and the coefficient of uniformity, respectively. In the same figures the maximum and minimum densities obtained for various samples of the same material, but with slightly different grain size distributions, are also presented for comparison. These figures show that most of the field samples are on the loose side.

A comparison between the field and the laboratory values of density versus slurry concentration is shown in Figure 6.6. Each point in this graph corresponds to an average of density determination of 3 to 28 samples. The scatter of both laboratory and field density data is caused by inaccuracy of the available methods of density determination (see Appendix C), by variations in grain size distribution from point to point due to the nature of the flow and by the variability of the flow itself, as discussed in Sections 3.5.4, 5.5.5 and 6.3. JBy inspection of the scatter of the grain size distribution parameters, one could have expected significant variability of density.

Density of granular materials depends on a number of factors such as grain size distribution, shape of grains, roughness of the grain surface, level of energy of the depositional environment and rate of deposition (Kolbuzweski, 1950; Gray, 1968, Mitchell, 1976, Allen, 1982, 1985, among others). All these factors affect the density of hydraulic fills, in a complex manner. For example, high flow rate of low concentration slurries causes an increase in beach density by creating a high energy environment associated with a low rate of deposition, but on the other hand it may also cause a decrease in density by accentuating hydraulic sorting and depositing a material with a lower coefficient of uniformity. In most cases it might be difficult to isolate and quantify all the interacting influences. As a result of all these factors affecting the material density, significant scatter exists on the data, which obscures some of the trends.

Figure 6.7 shows the steady state lines (Sobkowicz and Handford, 1990) for the TS sand that was utilized in the flume and field tests. The upper line was determined from samples with 11% fines and the lower one corresponds to 4% fines. Materials under a certain stress level that have a density such that the density/stress point plots above the steady state line have contractive behaviour and are liquefiable, while materials that plot below the steady state line have dilative behaviour (non-liquefiable). An average dry density was calculated for each field test and plotted in this figure at a low nominal stress level. The dotted lines drawn through each of these points correspond to the compressibility of the TS tailings sand measured in oedometer tests (data from Syncrude's files). The shaded area in Figure 6.7 corresponds to the range of values measured at Syncrude's tailings dam (Sobkowicz and Handford, 1990).

Figure 6.7 demonstrates that a hydraulic fill can in fact be placed denser than the steady state line (or non-liquefiable), if properly designed. It shows that it is possible to deposit a fill such that the initial density is below the steady state line and remains non-liquefiable up to significant stress levels. An optimization of the placement method can improve the overall density of the fill and affect the position of the fill density point in relation to the steady state line, i.e., the liquefaction potential of the deposit.

Therefore, it is important to understand the factors that control the density of hydraulic fills as optimization of the placement method can have clear benefits to the economics and safety of hydraulic fills.

Figure 6.7 also emphasizes the importance of the compressibility of hydraulically deposited sands, what has not been studied in detail so far. The value of compressibility used in Figure 6.7 corresponds to an average value obtained for deep samples, which may not be a fair representation of surficial samples. Factors such as fabric and grain size distribution should be taken into consideration when analyzing compressibility of hydraulic fills.

While some aspects of Figure 6.7 could be regarded as arguable (such as the value of the nominal stress level or the use of oedometric compressibility), the qualitative conclusion is still valid and it is of significant importance for the hydraulic fill technology. The main point is that a hydraulic fill may be liquefiable or not at a certain stress level depending on the depositional conditions during construction. Moreover, the placement method can be designed to maximize density in order to produced a non-liquefiable fill or to minimize any mechanical compaction that might be required.

6.6 - FABRIC OF HYDRAULIC FILLS

Fabric studies of undisturbed samples from the field deposition tests described in Chapter 5, and of undisturbed samples of the field material deposited in a laboratory flume (see Chapter 3) were carried out using a Scanning Electron Microscope (SEM). A detailed description of the procedure adopted is provided by Law (1991). Significant features of the samples that were analyzed are described in this section, and shown in selected micrographs. A quantitative determination of mean grain orientation was also performed and the results are discussed below.

6.6.1 - Qualitative Analysis

One of the most obvious characteristics observed in the micrographs is that field samples contain much more clay and silt size particles than the flume samples (Photos 6.1 and 6.2). In the field samples, clay particles are attached to the grain surface and form connectors between contiguous grains (Photos 6.1a to 6.1f). The fine material seems to be distributed throughout the sample, but especially near the sand grain contacts (Photo 6.1g). Even the areas of the field samples with the least amount of fines displayed some clay connectors between grains (Photo 6.1h). However, the laboratory samples contained little or no clay particles attached to grains and had generally clean contacts (Photo 6.2).

In many cases, these connectors found in field samples were formed mainly by clay particles as the connectors shown in detail in Photo 6.3. The contacts between clay particles are predominantly face-to-face contacts. Electron dispersive X-ray analysis indicated the the clay mineral in these connectors is probably illite. The clay mineral in the oil sands formation is illite or kaolinite (Dusseault, 1977). Smectite is found only in the upper few meters of the formation.

In other cases (Photo 6.4), these connectors seem to be agglomerates composed of a combination of clay, impurities and possibly bitumen. The presence of bitumen in these agglomerates is possible, because clay minerals (montmorillonite, kaolinite, illite and chlorite) adsorb bitumen on their external surfaces, forming clay-organic complexes (Czarnecka and Gillot, 1980). These complexes are less hydrophilic than the clay mineral itself and are stable enough to resist powerful organic solvents. Czarnecka and Gillot (1980) have also shown that the amount of bitumen adsorbed to form clay complexes depends mainly on the type of exchangeable cation on the clay and on the type of solvents present. In case these clay connectors prove to be favourable to the behaviour of tailings sand, this fact could be important in researching a method to enhance the formation, strength and stability of the connectors. All the connectors shown in Photo 6.4 seem to be agglomerate connectors. The structure marked as *C1* on Photo 6.4c seems to be the remains of one of these agglomerate connectors, that is still attached to one grain (*G*) but that had the other grain removed, possibly when breaking the specimen to expose a fresh face for SEM viewing. A close-up of the connector marked as *C2* on this photo is shown in Photo 6.4d.

The mechanism of formation of the connectors between grains may be associated with the localization of water menisci at the contacts when the material dries. All the fine particles in suspension are taken by the pore fluid to the grain contacts, leading to the formation of clay or agglomerate connectors at these points, depending on the type of material originally in suspension.

The presence of these clay connectors explains why field samples had enough strength upon drying to hold their shapes and withstand handling and preparation for SEM analysis. Laboratory samples (with few exceptions) did not show clay connectors

(Photo 6.2) and accordingly, did not have the same strength upon drying. This observation of less fines in laboratory samples compared to field samples was expected once the sand used in the laboratory experiments was brought from the field and sent to a drying plant where the drying process caused loss of fines. These observations compare well with grain size analyses carried out on nearby samples. Among the field samples, T1SG27 seems to have less clay than the average, while T2SG3 (taken from the bottom of a channel) seems to contain more clay than the average. The amount of fines passing the #200 sieve (0.075 mm), which includes silt and clay sizes, is 3.5% for T1SG27 and 11.6% for T2SG3.

The visual analysis of the micrographs also shows that this tailings sand is formed mainly by subangular to subrounded grains. Smaller grains tend to be more angular than larger ones (see Photo 6.5, for example). Larger, heavier particles are more susceptible to transport attrition than smaller grains since they tend to be carried by dragging, rolling and saltation, with high energy impacts. Smaller particles travel larger distances in suspension or by saltation involving longer trajectories. Therefore, larger grains tend to have their corners worn faster faster than smaller grains. Etching of the grain surface and solution pitting were also present (Photos 6.1e and 6.4a). Crystal overgrowth on quartz grains were relatively common as for example in Photos 6.1c and 6.4. Similar characteristics were observed by Dusseault (1977) studying the same material (McMurray formation) before going through the oil extraction process. It is interesting to note that many interlocked or interpenetrative contacts were observed, similar to the ones Dusseault (1977) described for the oil sand formation. Some examples can be seen in Photos 6.1, 6.2, 6.5 and 6.7. It seems to be a phenomenon that occurs both in the field and in the laboratory (flume and pluviation), facilitated by particular grain shapes. In most cases, interpenetrative contacts were found on vertical faces. Linear contacts were also observed (Photos 6.1f, 6.1g, 6.1h and 6.5).

Some bands of coarser or finer material were visible even at this small scale (Photo 6.6). They may form the basis of what is seen as a lamination on a macro scale.

During the qualitative analysis, it was observed that some micrographs seemed to have a large proportion of flat grain surfaces aligned with the face plane (Photo 6.8). Later when the photos were identified, these cases of planar or relatively flat faces with a number of grain surfaces parallel to the face seemed to coincide with horizontal faces of field samples. Although only speculative at this stage, it might be that these planar faces coincide with natural micro planes of weakness or preferred grain orientation. This characteristic was not observed in micrographs of flume samples. However, is was noted that both flume and field samples presented macro planes of weakness, so that they would easily break along these planes.

In micrographs of flume and field samples, there are several examples of arching (Photos 6.1c and 6.9), creating a fabric that has some very large voids and some areas of small voids.

The flow direction was inferred from the apparent alignment of grains and voids with no previous knowledge of the micrograph identification. An example of the apparent orientation of the grains can be seen on Photo 6.10. Comparing assumed orientations for different micrographs of the same sample, it was found that they were compatible. They were also reasonably comparable to the flow orientations recorded during sampling. These orientations will be compared with the quantitative analysis.

6.6.2 - Quantitative Analysis

Eighteen micrographs were selected for quantitative image analysis, representing the three faces of both flume and field samples: the horizontal face and two vertical faces, one parallel to the flow direction and one perpendicular to the flow direction.

The quantitative analysis was performed by using a manual digitizer connected to a microcomputer provided with an image analysis system with statistical capabilities . Almost 30 parameters can be measured using this system. The main five parameters selected for this study were:

a) *Area* - measures the area of the projection of the grain, i.e., the area of the grain as shown in the micrograph.

b) *Dellip-A* and *Dellip-B* - are the calculated lengths of the major and the minor axes, respectively, by approximating the grain shape by an ellipse.

c) *Form-ELL* - is the elongation factor of the calculated ellipse described above. It is defined by the ratio between *Dellip-B* and *Dellip-A*. It is, therefore, equal to 1 for a circle and zero for a line.

d) *Angle-AX* - is the angle between a reference line and the major axis of the calculated ellipse described above. It corresponds to the orientation of the axis in relation to which the momentum of inertia of the calculated figure is minimum. It varies from 0 to 180° and it is measured counterclockwise from the reference line.

The results of the quantitative analysis are summarized in Tables 6.2 and 6.3. The average area of particles measured from micrographs of horizontal faces was found to be larger than the area measured from vertical faces of the same sample for 10 out of 12 cases. And in the cases it was not larger, it was similar. The parameters *Dellip-A* and *Dellip-B* followed the same trend, with the values determined by horizontal faces being larger than the values found for vertical faces of the same sample. *Dellip-A* and *Dellip-B* compare well with the mean grain size (D_{50}) determined by sieve analysis on samples from the same location. In most cases, D_{50} corresponds to an intermediate value, smaller than *Dellip-A* and larger than *Dellip-B*, as expected.

The orientation of the grains long axes was analyzed statistically using the method proposed by Curray (1956). Rose diagrams of the orientation data were produced and are presented in Law (1991). The results of the statistical analysis were mostly in agreement with the orientations determined visually, and showed a certain degree of grain alignment in the directions parallel to the flow directions, in vertical and horizontal faces. Similar results were obtained for both flume and field samples.

6.6.4 - Comments

From the relatively limited analysis that was carried out, no significant differences between fabrics of flume and field samples were detected, except for the amount of fines and presence of connectors between grains. However the feed materials for the flume and the field tests did not have the same amount of fines and used different carrier fluids, which can explain the differences observed. Therefore, the fabric analysis seems to support the use of flume tests to reproduce the field structures, provided the same feed material is used.

6.7 - CONCLUSIONS

A distinction between segregating and non-segregating slurry behaviour is of importance for the study of hydraulic fills, because each type of slurry will form a fill with distinct characteristics. Segregating slurries deposit comparatively flatter and denser fills, with grain size distribution varying along the fill. Deposits built by non-segregating slurries have approximately constant mean grain size.

Flume tests have simulated adequately the flow conditions and the fabric that were observed in the field. The variation of mean grain size along the flume proved to be representative of the situation inside the channels of the field tests with mean grain size increasing towards the downstream. Evidences indicate that the sediment transport rate might be an important factor to be considered in the design of flume tests to model field conditions. Generally, flume tests seem to be able to reproduce, at least qualitatively the phenomena the occurs in hydraulic fills.

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TABLE 6.1 - Identified causes of fan head entrenchment (modified after Schumm et al., 1987)

	CAUSE	Selected References
1	Climate change towards more arid conditions	Lustig (1965)
2	Deglaciation	Funk (1976); Ryder (1971); Wasson (1977)
3	Regime change from predominance of mudflows towards more frequent stream flow conditions	Bluck (1964)
4	Regime change resulting in an increased frequency of mudflows	Lustig (1965)
5	Stream capture in the drainage basin resulting in increased discharge to fan	Dzurisin (1975); Eckis (1928)
6	Tectonism (uplifting and/or tilting)	Bull (1964); Hooke (1968)
7	Decreased load	Eckis (1928); Ryder (1971)
8	Increase in the frequency of high-magnitude rainfall events and decrease in frequency of low-magnitude rainfalls	Bull (1964)
9	Extreme event of intense rainfall and/or runoff	Beaty (1970, 1974); Bull (1964); Denny (1967)
10	Destruction of drainage basin vegetation, resulting in increased surface runoff	Bull (1964); Eckis (1928)
11	Alternation of debris flows and water flows	Hooke (1967)
12	Erosion of fan surface, headward gully erosion, followed by capture	Denny (1967); Rich (1935)
13	Lateral channel migration to steeper areas on the fan surface	Hooke (1967); Rich (1935)
14	Baselevel lowering (adjacent valley incision)	Drew (1873); Ryder (1971)
15	Toe trimming (valley stream encroachment)	Drew (1873); Wasson (1977)
16	Basin downwearing over geologic time	Eckis (1928)

TABLE 6.2 - Summary of Image Analysis Results

Sample	Photo #	Face	Area (mm^2)	DellipA (mm)	DellipB (mm)	B/A	% long grains	pseudo e	actual e	% < #200	CU	γ_d (g/cm^3)	D_{50} (mm)
Pluviated Samples													
FM 12	32/11	horiz	0.020	0.186	0.123	0.669	44.0	0.314	0.794	8.3	2.1	1.477	0.154
	33/14	vert 1	0.018	0.175	0.113	0.665	43.1	0.262					
	33/16	vert 2	0.019	0.180	0.120	0.665	43.3	0.369					
Flume Samples													
TS 34	46/37	horiz	0.024	0.208	0.138	0.669	41.7	0.127	0.850	10.2	2.2	1.432	0.178
	44/30	vert //	0.013	0.153	0.104	0.690	35.6	0.232					
	45/33	vert ⊥	0.022	0.211	0.128	0.631	54.5	0.217					
TS 36	13/38	horiz	0.015	0.159	0.105	0.670	43.7	0.127	0.906	11.2	2.4	1.390	0.178
	13/50	vert 1	0.011	0.155	0.091	0.603	63.6	0.171					
	13/54	vert 2	0.010	0.145	0.089	0.622	56.0	0.245					

TABLE 6.3 - Summary of image analysis results

Sample	Photo #	Face	Area (mm^2)	A (mm)	B (mm)	B/A	% long grains	pseudo e	actual e	% < #200	CU	γ_d (g/cm^3)	D_{50} (mm)
Field Samples													
F10/T2 SG3	60/78	horiz	0.016	0.166	0.109	0.666	41.2	0.391	0.761	11.6	2.60	1.505	0.151
	56/61	vert //	0.014	0.163	0.097	0.596	64.0	0.296					
	61/83	vert ⊥	0.017	0.176	0.113	0.653	53.2	0.419					
F6/T4 SG22	64/95	horiz	0.025	0.202	0.137	0.690	40.4	0.176	0.745	7.2	2.19	1.519	0.168
	65/101	vert //	0.023	0.201	0.135	0.678	40.2	0.303					
	06/02	vert ⊥	0.019	0.185	0.122	0.666	43.7	0.132					
F2/T1 SG6	63/92	horiz	0.015	0.163	0.101	0.634	53.1	0.352	0.713	11.1	2.72	1.547	0.163
	62/86	vert //	0.016	0.161	0.103	0.660	43.4	0.238					
	05/08	vert ⊥	0.012	0.145	0.087	0.613	52.4	0.337					

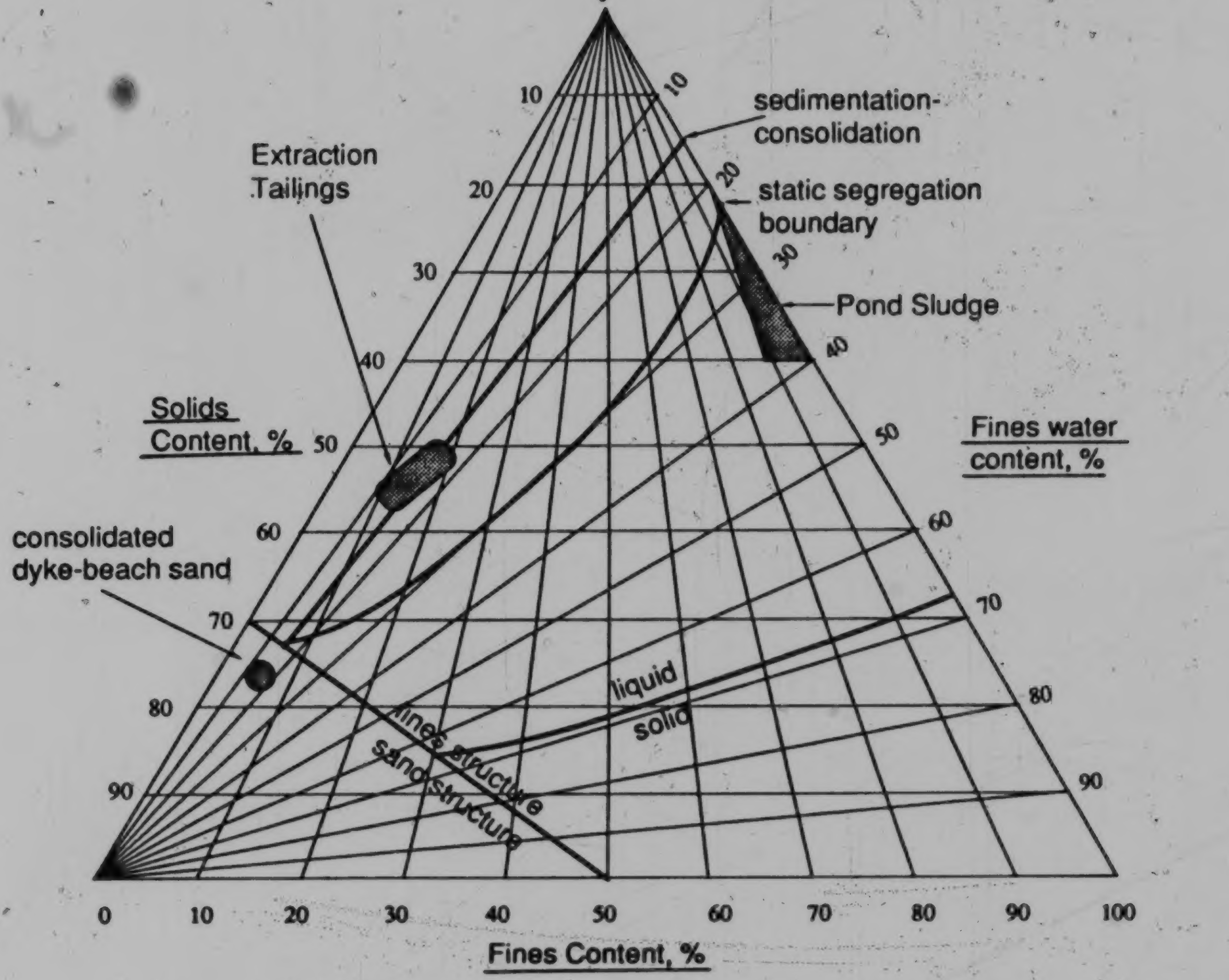


FIGURE 6.1 - Slurry behaviour diagram (modified after Scott and Cymerman, 1984)

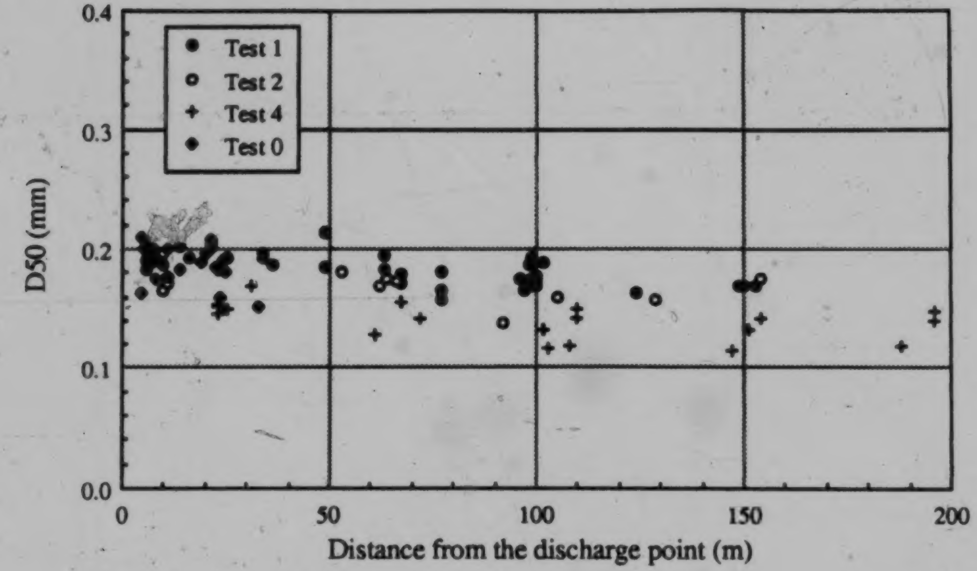


FIG. 6.2 - Variation of grain size distribution along the beach for the various tests

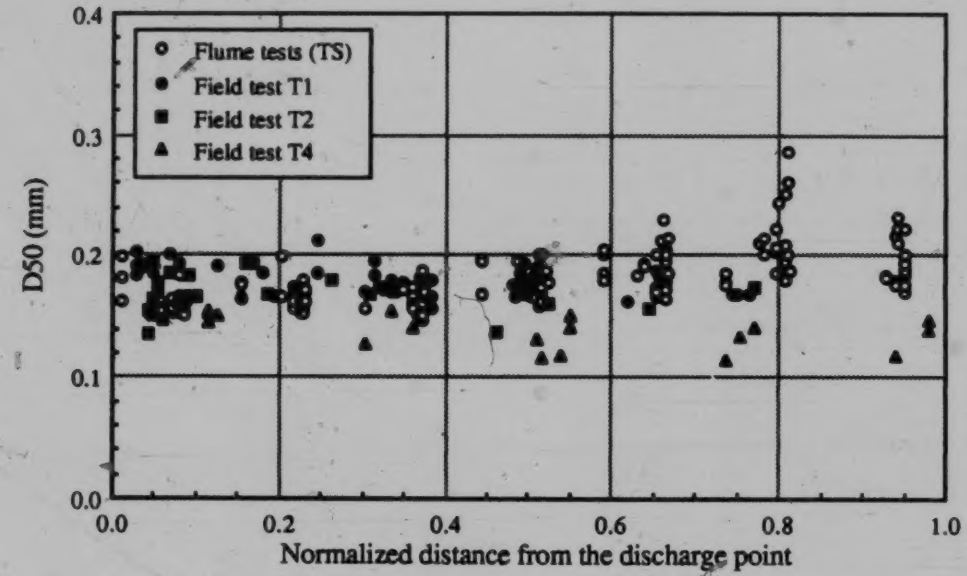


FIG. 6.3 - Variation of D50 with distance from discharge point (flume and field tests)

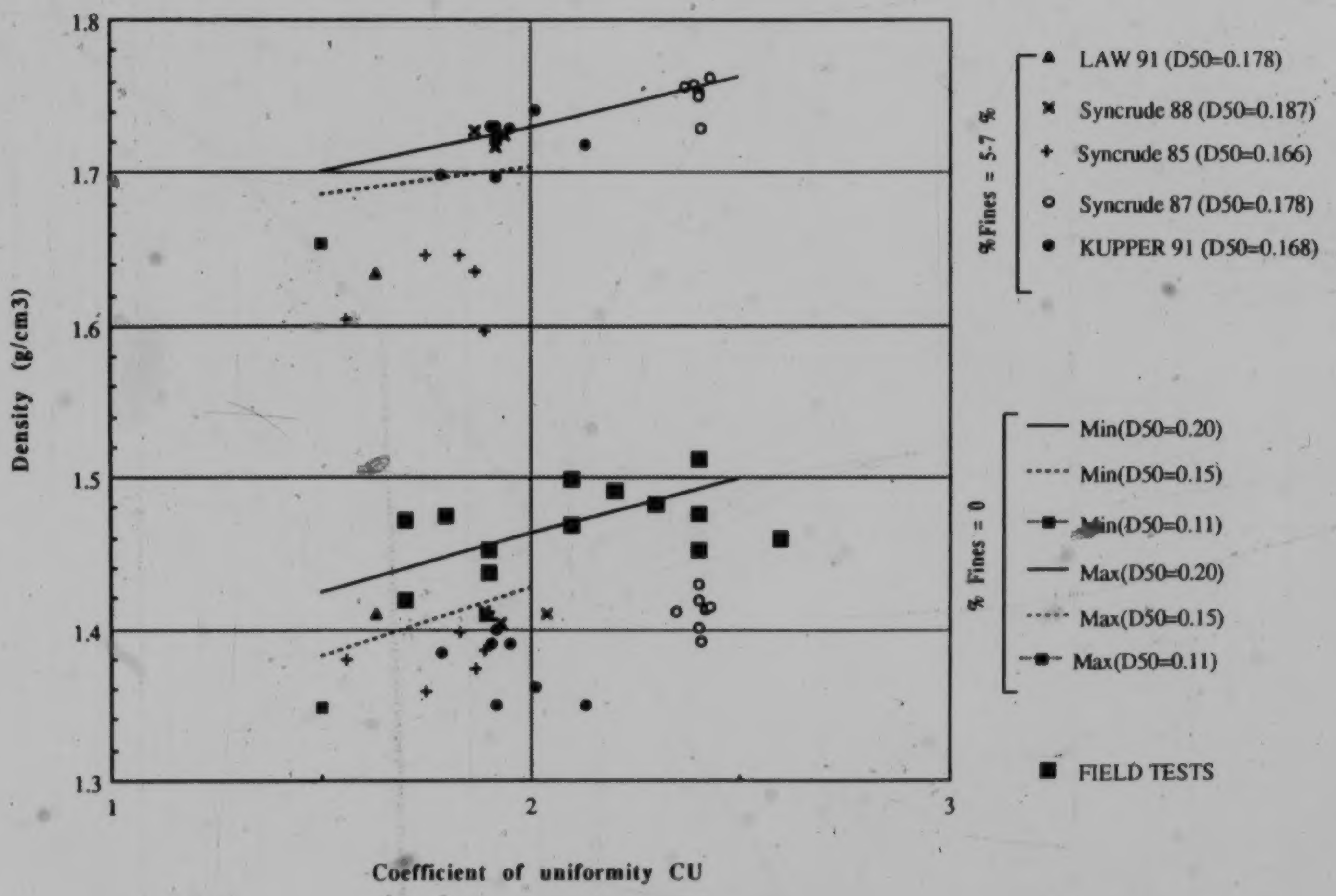


FIG. 6.4 - Density of the beach in relation to the maximum and minimum densities determined for the TS sand

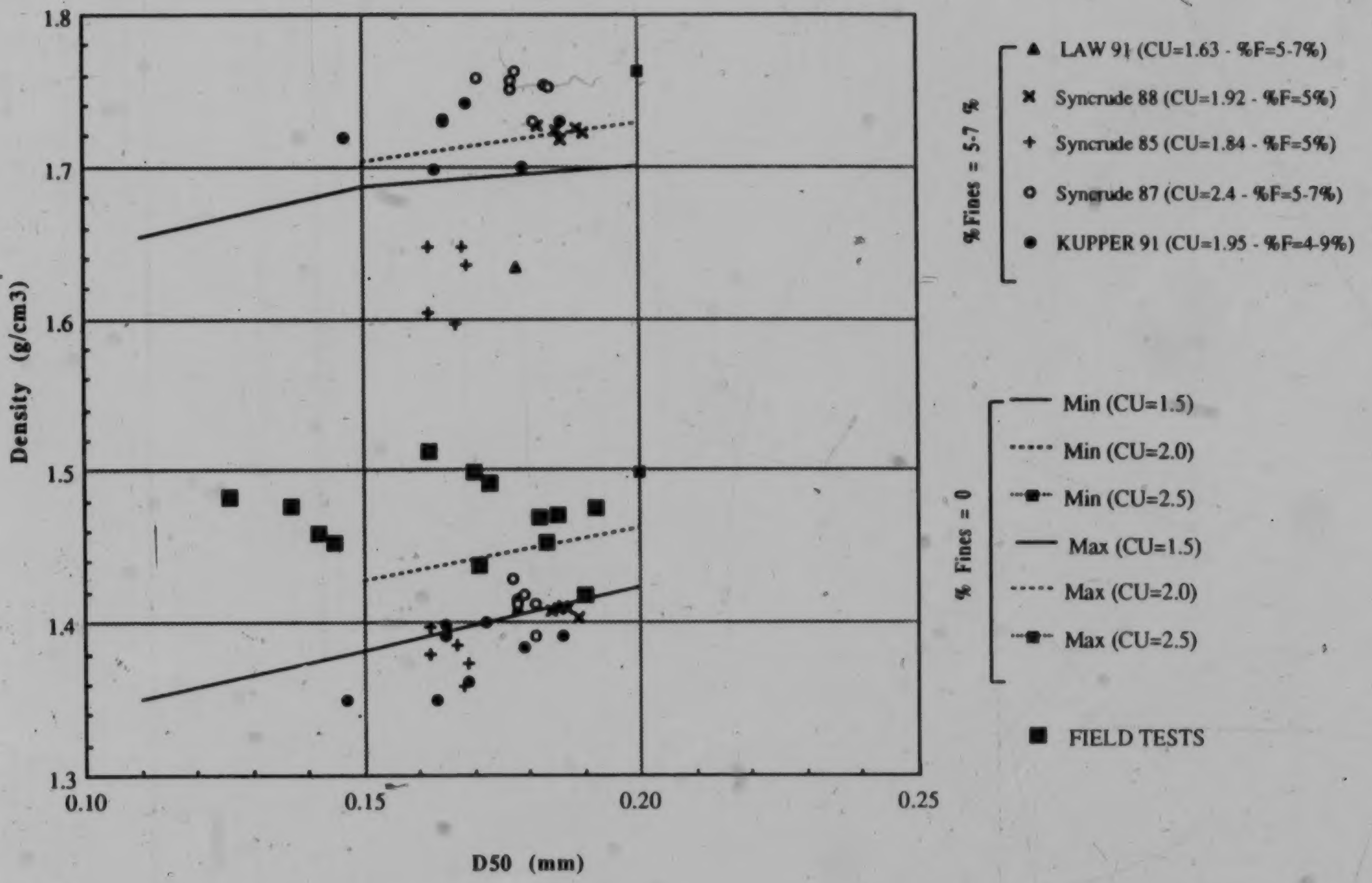


FIG. 6.5 - Density of the beach in relation to the maximum and minimum densities determined for the TS sand

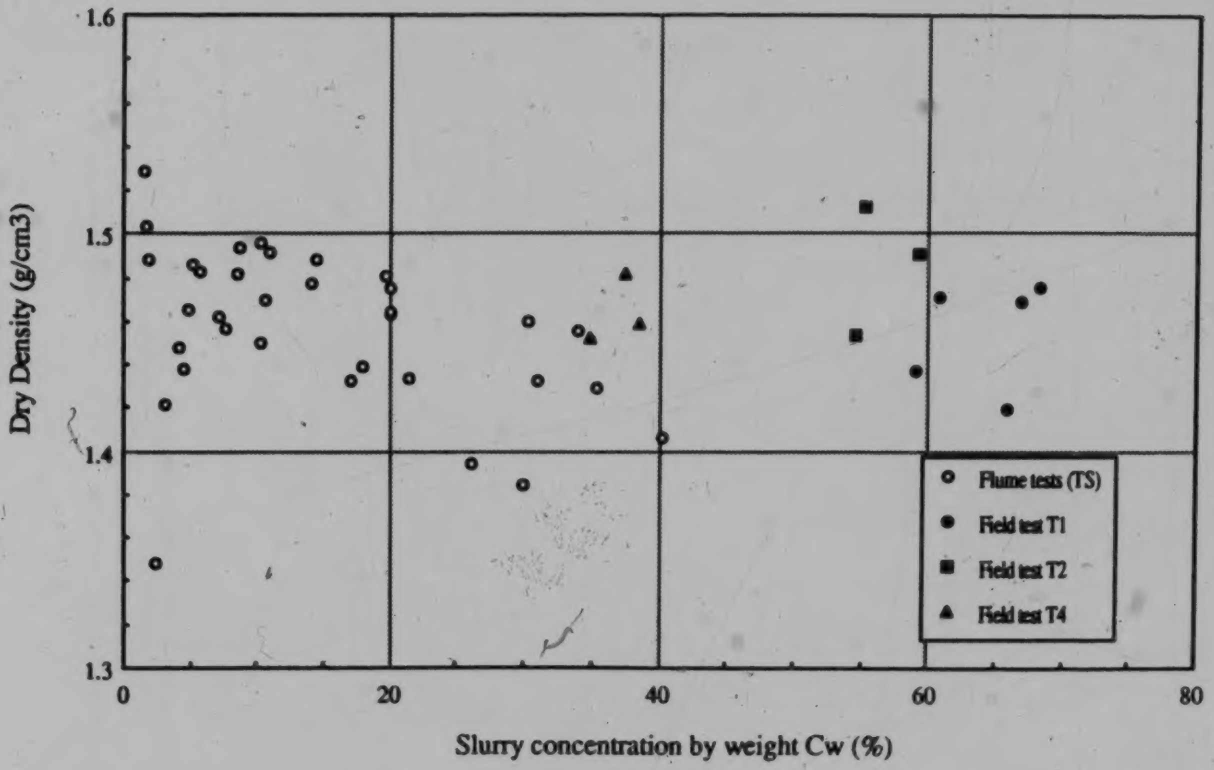


FIG. 6.6 - Variation of density with slurry concentration for flume and field deposition tests

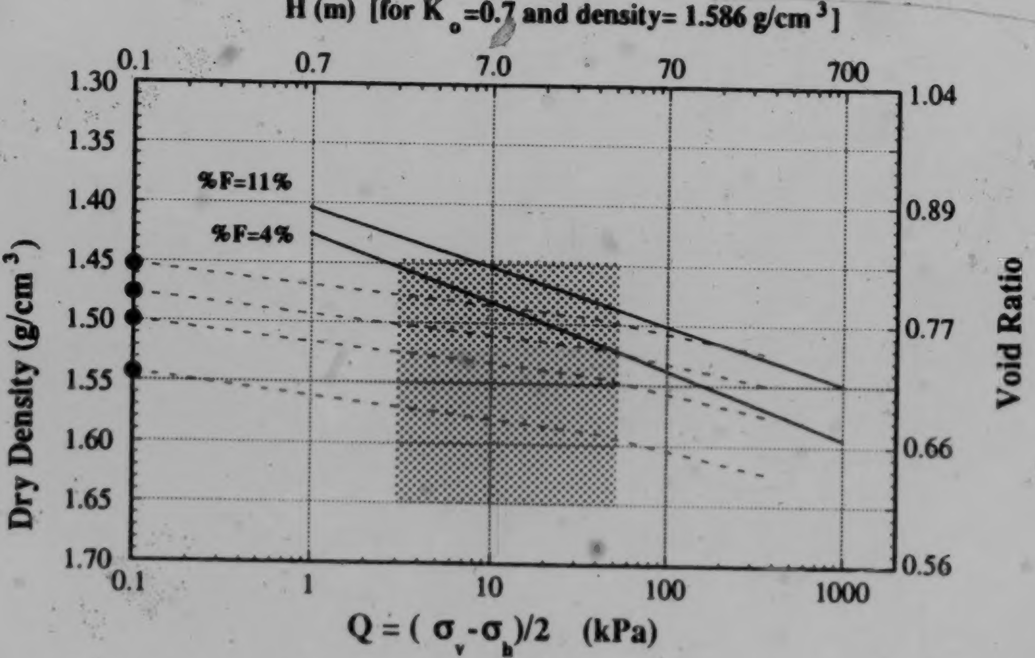
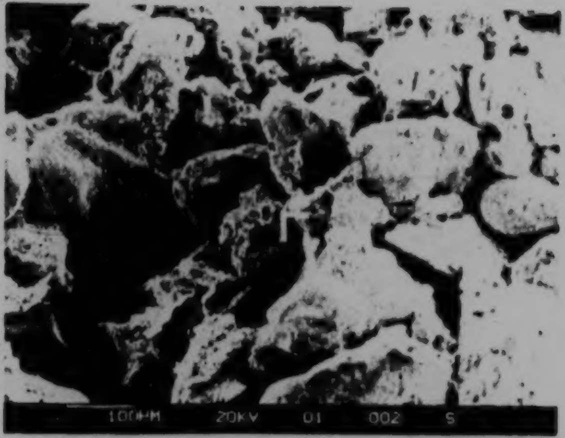
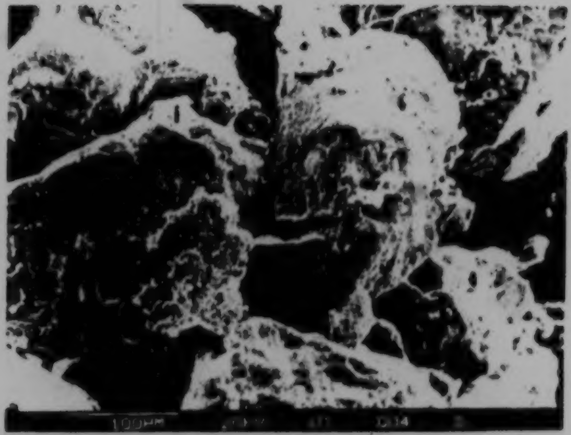


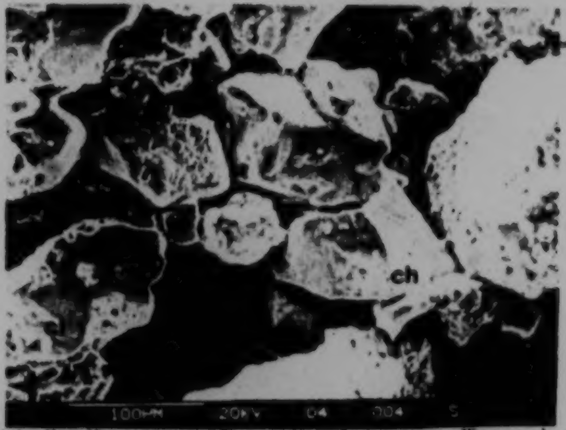
FIG. 6.7 - Average densities of the field tests in relation to the steady state line



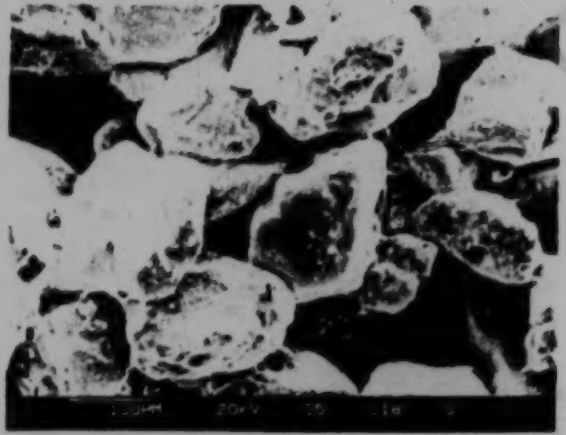
6.1(a) - Presence of clay particles attached to sand grains and forming connectors; contact point pressure solution concavity



6.1(b) - Clay particles attached to sand grains and forming connectors; interpenetrative contact (i)

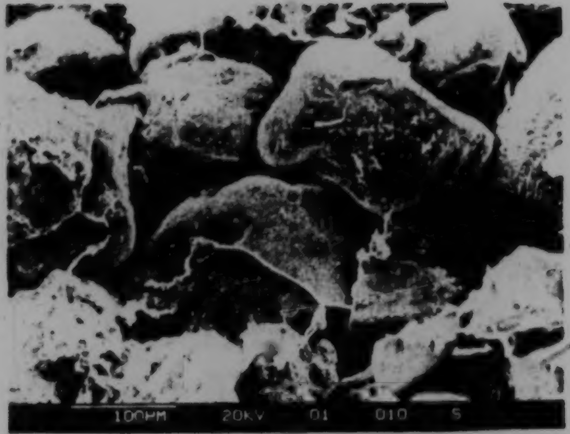


6.1(c) - Small connectors; interpenetrative contacts (i); remains of conchoidal fracture on quartz grain (ch); cleavage plane (cl); arching; crystal overgrowth visible on various grains

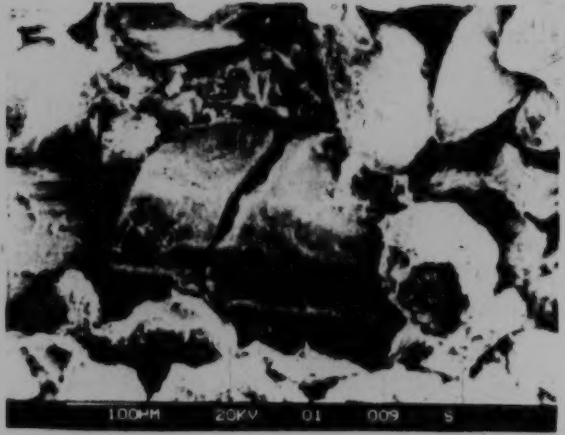


6.1(d) - Interpenetrative contact (i); connectors and clay particles attached to grains

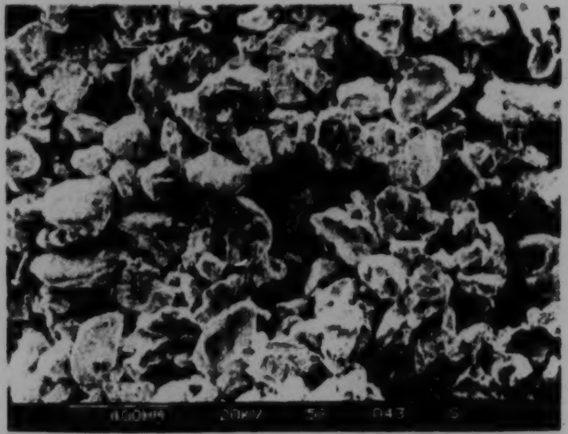
PHOTO 6.1 - SEM micrographs of field samples


100µm 20KV 01 010 S

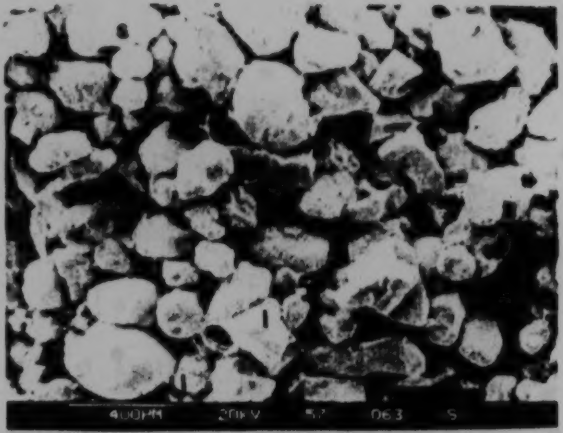
6.1(e) - Interpenetrative contact; connectors; contact point pressure solution concavity, solution pitting on grain surfaces


100µm 20KV 01 009 S

6.1(f) - Linear contact; clay connectors and clay attached to grains

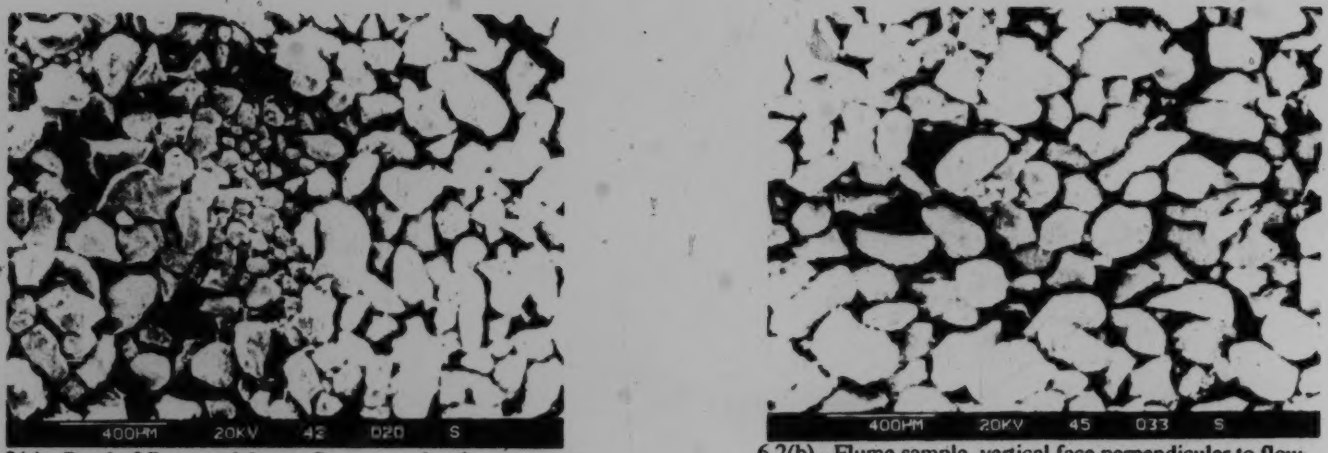

400µm 20KV 58 043 S

6.1(g) - Horizontal face; contact point pressure solution concavity (s); interpenetrative (i) and linear (l) contacts


400µm 20KV 57 063 S

6.1(h) - Horizontal face; contact point pressure solution concavity (s); linear (l) contacts

PHOTO 6.1 - SEM micrographs of field samples (Cont.)


400HM 20KV 43 020 S
400HM 20KV 45 033 S

6.2(a) - Band of finer particles on flume sample; clean contacts

6.2(b) - Flume sample, vertical face perpendicular to flow

PHOTO 6.2 - SEM micrographs of flume samples

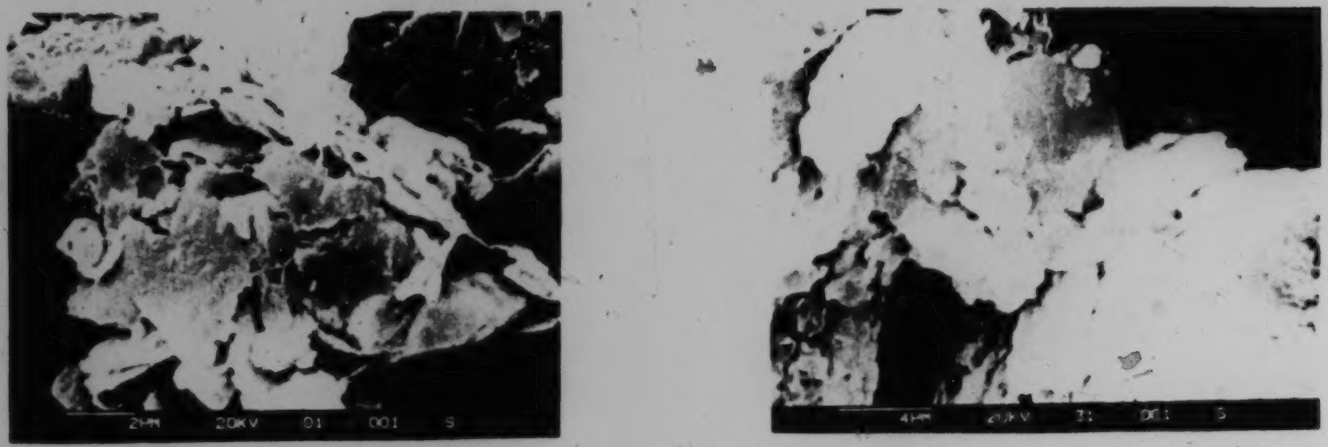
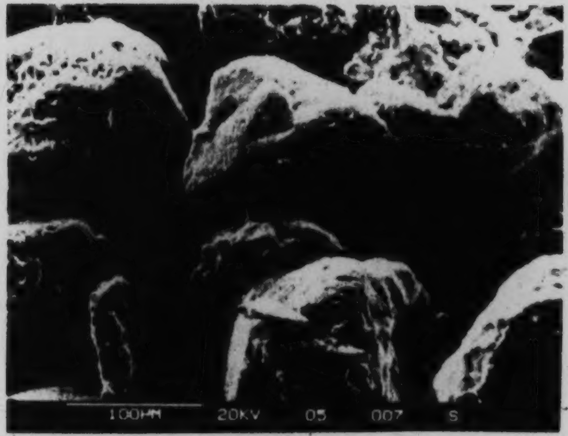
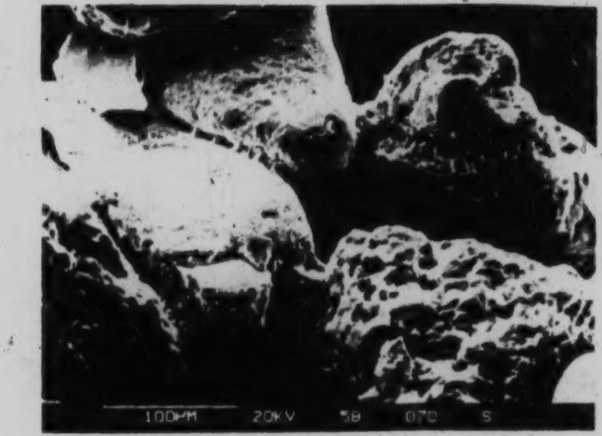

2HM 20KV 01 001 S
4HM 20KV 31 001 S

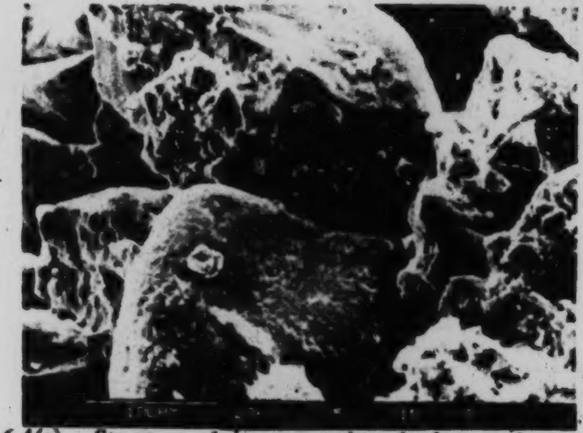
PHOTO 6.3 - Close-up of clay connectors


100μm 20KV 05 007 S

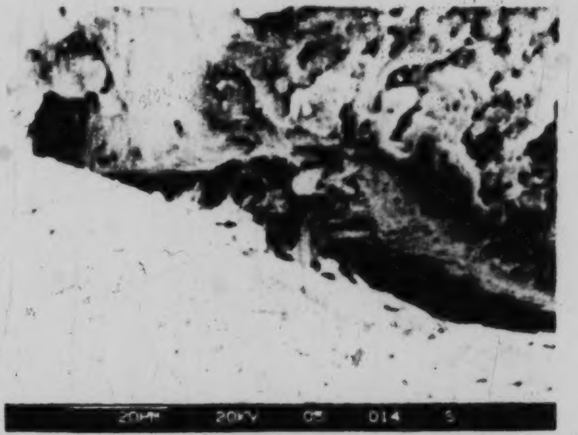
6.4(a) - Connectors, possibly constituted of clay-organic complexes; crystal overgrowth and solution pitting


100μm 20KV 58 070 S

6.4(b) - Connectors and crystal overgrowth on quartz grains



6.4(c) - Structure c1 is apparently a broken agglomerate connector, still attached to grain g, but with the other grain removed.


20μm 20KV 05 014 S

6.4(d) - Close-up of connector c2 shown in photo 6.4(c); crystal overgrowth on the top righ hand side

PHOTO 6.4 - Connectors between sand grains (field samples)

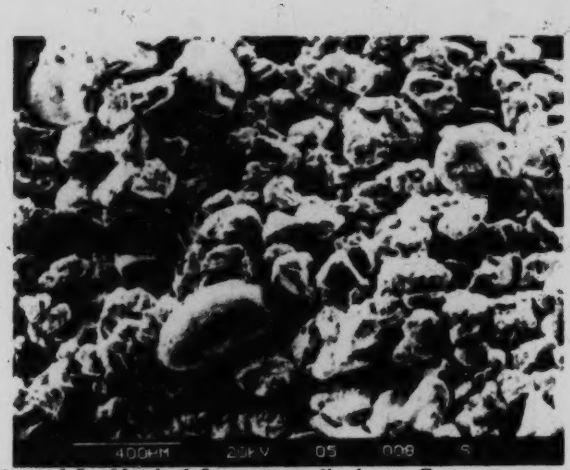

400µM 20KV 05 008 S

Photo 6.5 - Vertical face perpendicular to flow; connectors; linear and interpenetrative contacts (note grain angularity tendency to decrease for larger grains)

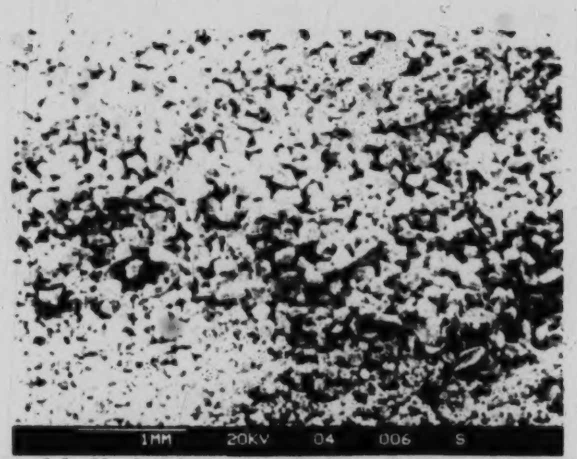

1MM 20KV 04 006 S

Photo 6.6 - Vertical face perpendicular to flow; note bands of finer material on the top and bottom parts; band of larger grains accross the centre of the photo is 1 to 2 mm thick

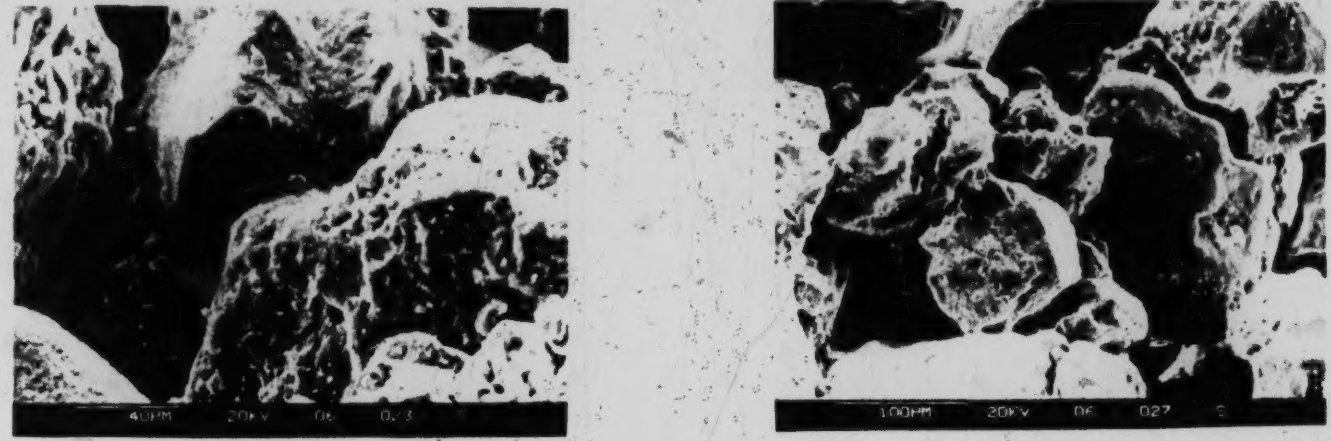
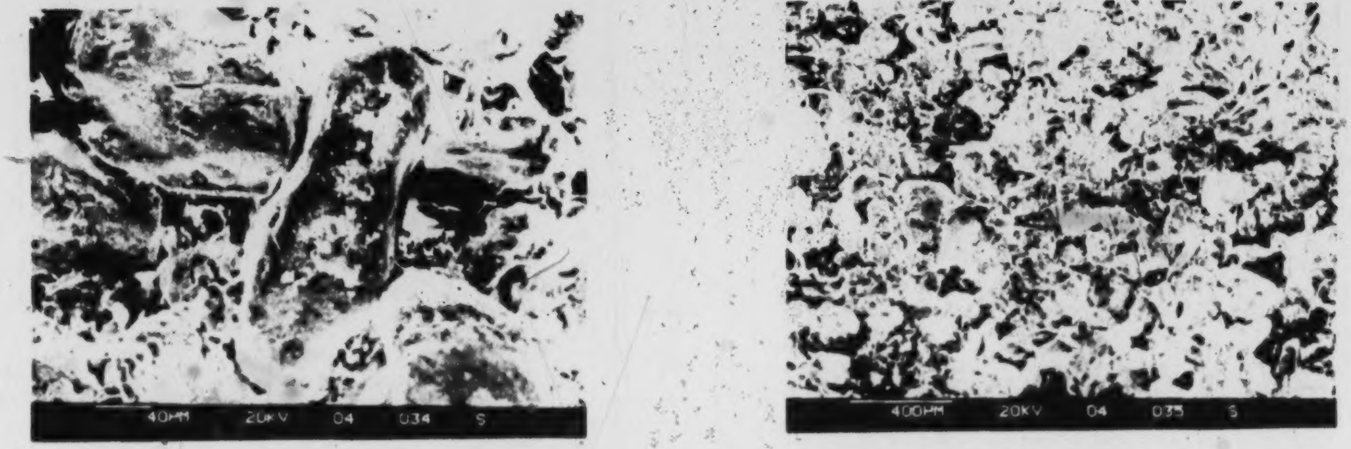
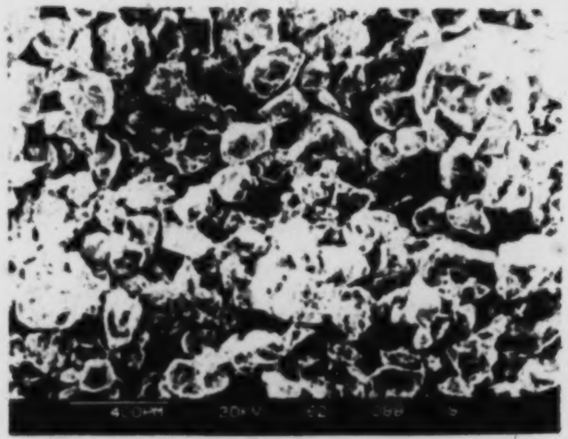

40µM 20KV 06
100µM 20KV 06 027 S

PHOTO 6.7 - Close-up of interpenetrative contacts on field sample T4 SG22



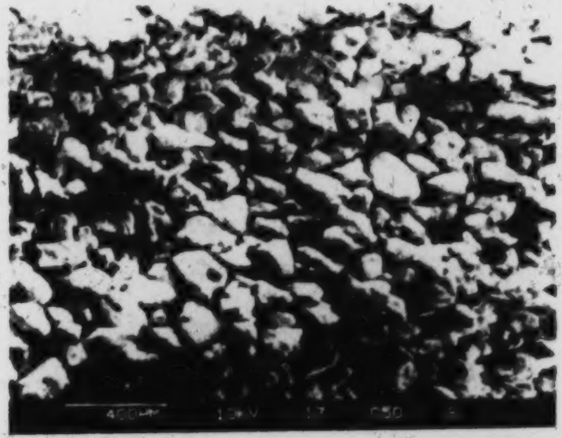
40HM 20KV 04 034 S
400HM 20KV 04 035 S

PHOTO 6.8 - Micrographs of sample T4 SG9 horizontal face



400HM 20KV 02 088 S

Photo 6.9 - Vertical face parallel to flow; arching creating larger voids



400HM 15KV 17 050 S

Photo 6.10 - Flume sample (between panels 2 and 3); note sand grain orientation

Chapter 7

Geometrical Considerations for Hydraulic Fill Beaches

7.1 - INTRODUCTION

Hydraulic fill beaches are formed by deposition of the solid fraction after the slurry is discharged. The properties of the beach, including its geometry, are a function of the characteristics of the material being deposited and of the discharge method.

The typical geometry of a hydraulic fill beach is a concave profile, steeper close to the discharge point and flatter further downstream (Figure 7.1). Both the average equilibrium slope and the concave shape of the beach are important factors in the design of hydraulic fill structures. The overall slope of the beach (Figure 7.1) determines the beach length and therefore it will influence the position and size of the pond (if there is one). The beach length is also significant for the location of water decant facilities. Consequently, the overall slope will have an impact on the lay-out of the whole structure as well as on the area to be occupied. Both area and lay-out are important issues in most projects. In the case of tailings dams, these factors may also affect other aspects of the mining operation such as access to the ore and mill location. In the case of subaerial disposal of dredged materials, which tends to be carried out along the shore line, the importance of the beach overall slope is usually related to limited area available and/or restrictions on the spill-box location. Although fill area and lay-out can be determined using only the average slope, both the average slope and the actual shape of the beach are necessary to calculate the volume of material deposited. Therefore both factors are necessary to estimate costs, duration of construction, size of the starter dam, lifetime of waste disposal facilities and storm water storage capacity. For waste disposal facilities, the ability to predict the actual beach profile also permits the estimate of the rate of rise of the beach crest and consequently the construction schedule. Clearly the beach geometry is an important issue in the design of hydraulic fill structures.

The objective of this chapter is to discuss the determination of beach geometry at the design stage. The factors that affect beach geometry are presented and the influence of each one is assessed in Section 7.2 and beach geometry is described in some detail in Section 7.3. Section 7.4 presents and evaluates the available methods to predict geometry, including flume tests. Finally in Section 7.5, a new parameter is proposed to predict the overall slope of a hydraulic fill beach as a function of the discharge parameters.

7.2 - FACTORS THAT AFFECT BEACH GEOMETRY

The geometry of hydraulic fill beaches is determined by the characteristics of the solid fraction, and by the discharge parameters such as slurry flow rate, slurry concentration and discharge velocity (see Chapters 3 to 5). This is also valid for materials deposited by flow in nature such as river beds (Gilbert, 1914; Mackin, 1948; Lane, 1955) and alluvial fans (Hooke, 1967, 1968).

Several flume tests carried out to study hydraulic fills were reviewed in Chapter 4. The results of these tests showed that the overall slope is a function of the discharge flow rate, slurry concentration and mean grain size of the solid fraction.

The larger the discharge flow rate, the flatter is the resulting slope. Figure 7.2 shows the decrease in overall slope with the increase in flow rate for the various flume tests discussed in Chapter 4. For the sake of clarity, the results of the experiments carried out by the Delft group (de Groot et al., 1988; Winterwerp et al., 1990) are not presented in this graph since the flow rates used were a couple orders of magnitude larger than the flow rates adopted for the remaining flume tests. Yet, the results obtained by the Delft group present the same trend shown by the other tests.

The overall beach slope becomes steeper as the slurry concentration increases and as the particle size increases. The variation of beach slope with slurry concentration for flume tests is presented in Figure 7.3. The slopes obtained on the large scale flume tests performed in Delft (DL1 and DL2) (de Groot et al., 1988; Winterwerp et al., 1990) are relatively low due to the very high flow rates that were used. Relatively flat slopes were also observed for the experiments carried out by the U.S. Bureau of Mines (USA and USB) (Boldt, 1988) due to the use of very fine materials. Therefore these sets of results also show the effect of the flow rate and the particle size on the slope. The effect of particle size on the overall slope can also by seen by comparing the results of KS and TS tests (Figure 7.3) that were carried out over the same range of flow rates and slurry concentration (Chapter 3). The coarser KS sand developed steeper slopes than the finer TS sand.

The same trends of variation of slope with slurry flow rate, concentration and particle size were obtained from field measurements. Figure 7.4 presents the decrease in overall slope for larger flow rates. The relatively larger scatter in the data presented in Chapter 5 is mainly due to lack of accuracy of the flow rate measurements and small variations in grain size distribution. The increase in slope with slurry concentration is shown in Figure 7.5.

The effect of the discharge parameters and sediment properties on the slope of natural deposits placed by flow is similar to the effects of these parameters on man-made hydraulic fills. The knowledge of the effect of discharge on slopes of streams is not new. Gilbert (1914) already discusses the fact that high discharges can carry a given sediment load on a lower slope than lower discharges could. Mackin (1948) made similar comments for alluvial rivers. Hooke (1968) used the argument that larger catchment basins will provide proportionally higher discharges to explain why fans with larger source areas developed flatter slopes. He also describes a case of reduction of slope of a natural fan after a creek was artificially diverted into the fan, increasing the flow rate. The influence of sediment size has been known to be important for the slopes of streams (Gilbert, 1914; Rubey; 1938 and Mackin; 1948) and alluvial fans (Hooke, 1967, 1968). Comparing alluvial fans on the east side of Death Valley, California, USA, Hooke (1968) observed that three fans that presented flatter slopes than other fans with similar drainage basin area, also had mean grain size that was less than the other fans. Also, two fans in Cactus Flats, California that are composed of material finer than most other fans, displayed a relatively flat slope in relation to the area of the respective drainage basins.

Hooke and Rohrer (1979) measured the slope at the axis of natural alluvial fans. Figure 7.6 presents a correlation between these slopes and the area of the drainage basin, which for hydrologically similar basins is directly proportional to the flow discharge at the basin outlet, i.e., the head of the fan (Chow, 1959). As the area of the drainage basin increases, and consequently the discharge, the slope decreases as expected. The only exception is Mauve Shadow Fan, however in this case the relatively flat slopes could be explained by the much smaller grain size and relatively high percentage of fines of the fan material.

These effects were readily observed in laboratory fans (Hooke, 1986; Hooke and Rohrer, 1979). The decrease in slope for smaller sediment size and larger discharge on the experimental fan slopes of Hooke and Rohrer (1979) is presented in Figure 7.7. The crossing of the curves on the lower side of Figure 7.7 was considered anomalous and was explained as being caused by boundary effects or by the fact that the equilibrium slope might not have being attained. Sediment concentration is implicitly assumed as constant since it was considered that "the sediment discharge at any given minute was proportional to the water discharge which acted during that minute". This is a crude assumption and possibly contributes to the crossing of the curves found in Figure 7.7.

The effect of sediment concentration on slopes of alluvial fans is discussed by Bull (1964) and Hooke (1968). As for man-made hydraulic fills, it was found that flows with higher sediment concentration build steeper fans, under otherwise equivalent conditions.

Lane (1955) proposes a qualitative relationship for analysis of stream morphology that summarizes the effects of flow rate, slurry concentration and particle size on all (man-made and natural) hydraulic fill slopes:

$$Q_s D_{50} \propto Q_f i \tag{7.1}$$

where:
Q_s - flow rate of sediment
Q_f - flow rate of fluid
D_{50} - mean grain size
i - bed slope

This relationship is consistent with the trends discussed above. It also predicts an increase in slope as the mean grain size and the flow rate of sediment increase and the fluid flow rate decreases.

The effect of the discharge velocity on the beach slope has not been directly discussed in this section. However, in most cases of hydraulic fill construction, velocity and flow rate are directly related since the discharge is usually carried out by constant section pipes flowing at full section. In this case an increase (or decrease) in flow rate is always associated with an increase (or decrease) in flow velocity. Both parameters cause the same type of effect on the beach slope, i.e., the slope becomes flatter as they increase and steeper as they decrease. For this reason the association of both parameters is acceptable for most practical cases. However, it would be of interest to differentiate between the effects of discharge velocity and flow rate in order to permit an optimization of the diameter and position of the discharge pipes.

There are other discharge parameters that have not been discussed here but that can also affect the fill slope. Parameters such as the height from the discharge pipe to the beach and the angle of the pipe in relation to the horizontal can change the velocity of the flow at the upstream end of the beach and therefore are bound to change the flow conditions on the beach and consequently the beach slope.

In summary, the above discussion concludes that for both natural and man-made hydraulic fills under either laboratory or field conditions, it has been repeatedly and consistently shown that the slope increases as sediment size and concentration increase and as flow rate decreases.

7.3 - DESCRIPTION OF HYDRAULIC FILL BEACH GEOMETRY

Hydraulic fill beaches develop a concave profile (Figure 7.1). The decrease in beach slope with distance is usually explained by the particle size separation that occurs on the beach (Melent'ev et al.,1973; Blight and Bentel, 1983, among others). The beach is steeper close to the discharge point where the larger particles usually are deposited, and flatter further down, where the finer particles tend to predominate, which is consistent with the stated effects of particle size on slope discussed above.

Particle sorting, although contributing to the beach concavity, may not be the main factor causing the concave shape. The flume tests described in Chapter 3 developed concave slopes, although the beach material was typically coarser towards downstream (see Figs. 3.33 to 3.35). The same effect was also seen in the field in some cases (Chapter 5). The profile along the bottom of large channels (as for example Profile I, Test 2) was concave (Figure 5.36) and the mean grain size increased with distance along the channel (see Figure 5.42). These examples demonstrate that, both under laboratory and field conditions, the concave shape of the beach is not caused mainly by the longitudinal variation in particle size along the beach.

Another factor that also contributes to the formation of a concave beach is seepage. Close to the discharge point there is infiltration into the slope. This downward component of the flow causes an increase in the relative weight of the particles, that will cause particles to settle sooner than they would otherwise. However, due to the high sediment concentration in this region this effect might be somewhat counterbalanced by hindered settling. At the downstream end of the beach there is flow coming out of the slope increasing particle buoyancy, which contributes to a flatter slope.

However, the main factor responsible for the formation of a concave slope seems to be the variation of sediment concentration in the bed load layer. Closer to the discharge point, the sediment concentration in the flow is high, which produces steep slopes. Further downstream, after some excess sediment load has been deposited, the concentration becomes progressively smaller and consequently the slope becomes flatter.

Based on fluid mechanics and a sediment transport relationship, Fan (1989) derived a form of the diffusion equation that can be used to describe beach profiles. This equation does not consider seepage or variations in particle size, and yet, for the boundary conditions associated with hydraulic filling, the solution of the equation is a concave curve. Therefore, although particle sorting and seepage may contribute to the concave shape, they are not its main cause.

For non-segregating slurries, the physical phenomena involved are different. The flow does not create a bedload layer as defined in classic sediment transport theories. The mechanics of sediment deposition under these circumstances are different and no sediment sorting occurs with distance from the discharge point. For flume tests with non-segregating slurries, Fourie (1988) obtained convex upwards profiles.

According to Melent'ev et al.(1973), the concave profile of hydraulic fill beaches can be described by the following dimensionless equation:

$$\frac{y}{H} = \left(1 - \frac{x}{L}\right)^{n} \tag{7.2}$$

where L is the beach length, H is the maximum beach elevation (see Figure 7.1) and n is a parameter that depends on the material and on the deposition method.

Several field and flume measurements confirmed this equation to describe both field and laboratory beaches adequately (Blight and Bentel, 1983; Blight et al.,1985; Smith et al., 1986; Fan, 1989). These authors determined values of n varying between 1.2 and 4, depending on the type of material being deposited.

Smith (1984) and Smith et al. (1986) showed that beach profiles can also be well described by an exponential equation of the form:

$$\frac{y}{H} = a \exp\left(b \frac{x}{L}\right) \tag{7.3}$$

where a and b are regressive constants. This equation should be modified to:

$$\frac{y}{H} = \exp\left(b \frac{x}{L}\right) \tag{7.4}$$

since for $x = 0$, by definition y has to be equal to H (Figure 7.1). In fact, Smith (1984) obtained values for a close to 1.

The slope varies along the beach, decreasing with the distance from the discharge point. This local slope can be calculated by differentiating equation (7.2):

$$i\,(x) = n\,\frac{H}{L}\left(1 - \frac{x}{L}\right)^{n-1} \tag{7.5}$$

An average beach slope could be defined as:

$$\bar{i} = \frac{1}{k}\sum_{j=1}^{k} i(x_j) \tag{7.6}$$

or simply as the slope at $x = L/2$. However in most cases $n \approx 2$ and for either definition the average slope will be

$$\bar{i} = \frac{H}{L} \tag{7.7}$$

The overall slope $(i_{ov} = H/L)$ is a convenient average slope since it is easy to determine, even when the actual profile is not known, and it will be adopted in this paper/chapter.

A rough estimate of the parameter n can be obtained in the field by measuring the slope close to the discharge point $i(x=0)$, provided this region has not been significantly disturbed by the plunge pool. In this case the parameter n will be:

$$n \cong \frac{i\,(x=0)}{i_{ov}} \tag{7.8}$$

This type of estimate might be of use in cases where the lower part of the beach is too soft and it is only possible to walk on its upper part.

When slurry discharge proceeds for long enough under constant conditions, the overall slope does not change considerably from one time to another. This has been observed in laboratory experiments (Soni, 1981; Fan, 1989; Chapter 3) and in the field

(Melent'ev et al., 1973; Wang et al., 1983; Chapter 5). This slope is called the equilibrium slope, in the sense that it is in equilibrium with the discharge parameters. If the slurry concentration is increased, for example, the flow will steepen the upper part of the slope, which will progress gradually downstream until an new steeper equilibrium slope is established. A similar adjustment to the slope occurs when the slurry concentration is decreased or the flow rate is increased: the flow erodes the upper part of the deposit and deposit on the lower part until the overall slope is flattened to the new equilibrium slope compatible with the new input parameters.

de Groot et al.(1988) and Winterwerp et al.(1990) adopt the definition of equilibrium slope used in sediment transport studies. In this case, equilibrium slope corresponds to the slope at which sedimentation and erosion are in equilibrium, which means that the sand bed is neither aggrading nor degrading. However, hydraulic fill beaches constitute a depositional environment that is continuously aggrading, but in such a way that each new beach surface tends to be approximately parallel to the previous one.

7.4 - AVAILABLE TOOLS TO PREDICT BEACH GEOMETRY

7.4.1- Shape

Given the importance of the beach geometry for planning and design of hydraulic fill structures, it becomes necessary to have design tools to determine geometry as a function of the discharge parameters.

An analytical method to predict the geometry of hydraulic fill beaches was developed by Fan (1989), based on previous work developed for alluvial stream beds. A non-linear parabolic model is proposed for the aggradation of an alluvial bed that permits one to calculate the beach elevation at any location as a function of time (Fan, 1989; Fan and Masliyah, 1990). A quasi-steady and uniform flow condition is assumed and the governing equations adopted are the continuity equations for fluid and sediment, momentum equation for fluid, Manning's equation for flow resistance and Meyer-Peter and Muller bed load formula for sediment transport. A numerical solution is obtained by finite differences, using an explicit scheme. This method produced an excellent agreement between the numerical solution and the experimental results obtained by Fan (1989). The experiments consisted of flume deposition tests using a D_{50} = 0.267 mm sand and low slurry concentrations (8 to 14%). There was no water impoundment at the downstream end of the flume and the tests were terminated before the deposit reached the end of the flume. The results were reviewed and compared with other flume tests in Chapter 4.

This is an interesting approach since it is based on the equations that describe the physical phenomena involved and allows one to calculate the elevation along the entire beach, so both shape and overall slope are determined. However, at the present stage this method is not readily applicable to field conditions, for two main reasons. Firstly, it does not consider the existence of a water body at the downstream end of the beach, assuming that all the sediment is deposited on the beach and that the beach length grows indefinitely. Secondly, the method has been developed for slurry concentrations many times smaller than typical field values. A modification of the sediment transport equation and the use of different boundary conditions might enable this method to predict actual beach profiles, at least up to the value of slurry concentration for which the flow does not segregate However, these modifications are out of the scope of the present work and are left as recommendations for further research.

Another approach consists of scale modelling beach deposition in a laboratory flume. In the laboratory, the physical phenomena can be simplified by isolating individual variables. Observations and measurements can be made more easily and at a reasonable cost. In fact, laboratory flume tests have been instrumental in determining the factors that affect beach geometry as discussed in Section 7.2. Flume tests also made possible the study of the influence that each variable can have on beach characteristics. However, flume tests have proved not to be suitable to predict quantitatively some properties of field scale beaches. For example, both flume and field deposition produce concave beaches that are similar in shape, but laboratory beaches tend to be much steeper than their field counterparts as shown in Figs. 7.8 and 7.9 for two different sets of flume and field results. Figure 6.8 presents the results of field tests (Chapter 5) and the results of flume tests using the same material (Chapter 3). The field results reported by Winterwerp et al.(1990) are shown in Figure 7.9 along with results of the DL2 flume tests (see Figure 7.3) which utilized sand with mean grain size similar to the field sand. This discrepancy between flume slopes and field slopes precludes the direct use of flume tests results to predict actual beach slopes.

Blight et al.(1985) showed the use of flume tests to estimate the dimensionless profile (as in Equation 7.2) of a gold tailings dam. By using the same material and the same slurry concentration as in the field, Blight et al.(1985) obtained a flume profile that is described by the same parameter n on Equation 7.2 as the field profile. They conclude that flume tests could be used to determine the "master profile" of the beach. But in order to predict the actual field profile, it is still necessary to estimate the overall beach slope H/L for field conditions, which cannot be achieved by flume tests. Also, Blight et al.(1985) make no comment on which value of flow rate should be adopted for the flume tests.

Fourie (1988) carried out flume tests with non-segregating slurries of three different tailings materials. He also found out that Melent'ev's equation normalizes well different profiles, but as opposed to Blight's findings, Fourie concluded that the normalized profile does not depend on slurry concentration (or type of fluid). Unfortunately, flow rate was kept constant during Fourie's tests and field profiles were not presented for comparison. It was also observed that the normalized profile is sensitive to particle size distribution. Coarse coal tailings yielded a normalized profile that was very different than the profile for fine coal tailings under similar conditions. However, it may also be that the normalized profile is sensitive to slurry behaviour (instead of or as well as particle size), as the coarse tailings slurry was a segregating slurry while the fine tailings was non-segregating.

The approach of studying geometry by obtaining normalized profiles from flume tests was evaluated using a larger data set of flume and field results Chapters 3 and 5, respectively). Both flume and field tests utilized the same material. Thirty seven flume tests were performed for flow rates varying between 80 and 330 x 10^{-6} m^3/s and slurry concentrations between 1.5 and 40.5%. Equation 7.2 was numerically adjusted to all the profiles obtained and yielded *n* values varying between 1.04 and 1.67, with an average of 1.35 and coefficient of variation of 10.7% ($0.96 \leq r^2 \leq 1.00$). A typical case is presented in Figure 7.10. The values of *n* obtained for these tests do not seem to correlate well with either slurry concentration or flow rate (Figure 7.11). Similar treatment was given to eighteen field profiles that were deposited by slurries with concentration varying between 35 and 68%. Estimated flow rates were in the range of 0.15 to 0.98 m^2/s. An example of the curve fitting for the field profiles is given in Figure 7.12. The values of *n* varied from 1.19 to 2.13 with an average of 1.49 for the cases where the spigot diameter was 15 cm (larger flow rates) (Figure 7.13). For the cases of 7.5 cm spigots (smaller flow rates), *n* ranged between 1,74 and 2.60 with an average value of 2.26. The values of *n* obtained for the field profiles do not seem to depend on slurry concentration, but might have been influenced by the flow rate, as shown in Figure 7.13. This figure also presents the *n* values obtained from the flume tests, for comparison.

The limited amount of data presented by Blight et al.(1985) suggests that *n* increases with mean grain size (and coefficient of uniformity), however no correlation between *n* and grain size distribution parameters was found analyzing the data presented in the previous chapters. This lack of correlation substantiates the argument on Section 7.3 that grain sorting is not the main factor leading to beach concavity.

The difference between the dimensionless profiles described by the flume results (*n*= 1.35) and by the field results (*n* = 1.49 and *n* = 2.26) is presented on Figure 7.14.

Flume tests could have been used to provide a reasonable estimate of the dimensionless beach profile for the tests with the 15 cm spigot, but were not adequate to estimate the dimensionless profile of the 7.5 cm spigot tests. Therefore, it is still not possible to extrapolate flume data to the field with confidence, as preliminary results from Blight et al.(1985) first suggested.

The analysis of the data presented in Chapters 3 and 5 leads to a similar conclusion as Fourie (1988) that, although flume tests have been shown to be useful in studying the variables that affect hydraulic deposition, a number of questions remain in relation to their use for design estimates. It seems that there is not enough understanding of the factors that affect *n* at this stage to use flume tests to predict the shape of field beaches. It must also be pointed out that even if this method could predict actual field profiles, it would still require the knowledge of the overall slope for the field conditions, which cannot be determined directly from the flume results. Therefore, a method to estimate overall slopes at the design stage is also necessary.

7.4.2- Overall slope

Blight and Bentel (1983) state that at any point along the beach, the slope can be calculated by the equation of the stability of an infinite slope. Assuming seepage parallel to the slope, purely cohesive material and a factor of safety equal to 1, they obtained:

$$i = \arcsin \frac{\tau_0}{\gamma\,\delta} \tag{7.9}$$

where: τ_0 is the shear strength of the material just after settling
γ is its unit weight and
δ is the thickness of the material having shear strength τ_0

For a frictional material the expression for an infinite slope with parallel seepage and factor of safety equals to 1 is:

$$i = \frac{1}{2} \arctan \phi \tag{7.10}$$

where ϕ is the angle of repose of the slope material.

However, equations (7.9) and (7.10) do not consider that there is a flow of depth d over the slope and this flow applies a shear stress τ on the surface of the slope that is given by (Vanoni, 1975, p.75; Yalin, 1977, p.21):

$$\tau = \gamma_w \, r_h \sin i \tag{7.11}$$

where r_h is the hydraulic radius.

With these additional considerations, the expression for the angle of an infinite slope with seepage parallel to the slope and factor of safety $FS = 1$ becomes:

$$\tan i = \frac{\gamma_t - \gamma}{\gamma_t + \gamma \frac{d}{h}} \tan \phi \tag{7.12}$$

where: γ_t is the unit weight of the deposited material
γ_f is the unit weight of the fluid flowing over the slope
d is the depth of flow over the slope
h is the thickness of the layer for which $FS = 1$

These expressions are valid only in the case of an existing slope subjected to the conditions for which each equation has been derived and do not apply for slopes that are being formed under the specified conditions. The slopes obtained from these expressions could be used as an upper bound value for the actual beach slope, but do not represent the slope at any point along the beach. Nevertheless, even the use of these expressions to obtain an upper bound value of the slope is difficult as the values of δ and h cannot be obtained easily. Moreover, it is not clear that hydraulic fill slopes should have $FS = 1$ during deposition.

Introducing hindered settling into the sediment transport equation proposed by Engelund and Hansen (1967), de Groot et al. (1988) propose the following equation for the angle of hydraulic fill beaches:

$$\tan i \propto D_{50}^{0.6}\, C_v^{0.6} (1 - C_v)^{12}\, q^{-0.4} \tag{7.13}$$

where: C_v is the slurry concentration in terms of volume
q is the specific flow rate (flow rate per unit width)

This equation is further developed in Winterwerp et.al.(1990) becoming:

$$\tan i = \left[\frac{(G - 1)^2 D_{50}}{0.20} C_v (1 - C_v)^2\right]^{0.6} \left[\frac{g^2}{8/f_0}\right]^{0.1} q^{-0.4} \tag{7.14}$$

where f_0 is the Darcy-Weisbach coefficient, which for the field and flume experiments presented by Winterwerp et al.(1990) varied between 0.04 and 0.15 (f_0 was 0.53 for the small scale flume tests).

Equation (7.14) was used to evaluate the results of flume and field tests. As shown in Figure 7.15 this equation produced reasonable results for the TS tests that developed the flatter slopes (< 4%), but not for the remainder. The prediction was less adequate for the field tests (FT) and for the KS flume tests (Figure 7.16). Similar results were obtained for the other sets of field and laboratory data discussed before (Figure 7.17). Equation (7.14) also does not predict well the slopes produced by the deposition tests presented by Winterwerp et al.(1990), as shown in Figure 7.18.

An empirical equation derived from large scale flume tests and field measurements is also presented by de Groot et al.(1988) and Winterwerp et al.(1990):

$$i_{av} = \left[0.0056 \; \frac{D_{50}}{65 \; \mu m} - 0.0045\right] \left[\frac{q}{1 \; m^2/s}\right]^{-0.45} \tag{7.15}$$

This empirical equation is only valid for $D_{50} > 65 \; \mu m$ and for $q > 0.01 \; m^2/s$, and therefore does not apply to any flume data from the literature. The application of this equation to the results of field tests presented in Chapter 5 led to calculated values of slopes that were 3 to 5 times smaller than the observed slopes.

Therefore, the conclusion from the above discussion is that the master profile equation of Melent'ev et al (1973) seems to provide a reasonable approximation of the beach profile shape, however it is not as simple to obtain the master profile from flume tests as it appeared from Blight's tests. The influence of several factors on the value of the exponent n is not well understood to this point. In any case, the utilization of the master profile concept still depends on knowing the overall slope. From the analysis of the equations for overall slope presented on the literature, the conclusion is that there is still no adequate method to predict the beach overall slope at the design stage.

7.4 - A DIMENSIONLESS PARAMETER TO ESTIMATE THE OVERALL SLOPE OF HYDRAULIC FILL BEACHES

The study of the geometry of hydraulic fill beaches based on the physical principles that govern the deposition process seems to be the ideal approach for the understanding of the phenomena involved. However, the development of this type of approach has not reached a stage where the equations could be used confidently in practice to predict beach slopes. Hence, a more practical approach to the question of slopes of hydraulic fill beaches will be taken here.

Based on the discussion on the factors that affect the overall slope of a hydraulic fill beach (Section 7.2), the following functional relationship can be written:

$$i_{av} = f\left(C_w, D_{50}, g, \frac{r_s - r_w}{r_w}, \frac{1}{Q}, A\right) \tag{7.16}$$

where: C_w is the slurry concentration in terms of weight [ ]
D_{50} is the mean grain size [L]
g is the acceleration of gravity [L T^{-2}]
ρ_s is the specific weight of the grains [F L^{-3}]
ρ_w is the specific weight of the water [F L^{-3}]
Q is the total flow rate at the discharge point [L^3 T^{-1}]
A is the area of the discharge pipe [L^2]

Using the variables listed above, and based on the trends of variation of overall slope with these variables as observed in the experiments (Chapters 3 to 5) and on the format of other sediment transport parameters, the following relationships are proposed:

$$P' = \frac{A\sqrt{g(G-1)}\sqrt{D_{50}}C_w}{Q} \tag{7.17}$$

where:

$$G = \frac{\rho_s}{\rho_w}$$

and

$$i_{av} = f(P')$$

The dimensionless parameter *P'* represents the ratio between gravitational forces and inertial forces. The relationship between the beach slope and the parameter *P'* is based on the concept that the larger the inertial forces are in relation to the gravitational forces, the farther the grains could be transported and consequently, the flatter the slope will be. This parameter is a modified Richardson's number multiplied by the slurry concentration. Richardson's number is actually the inverse of the densimetric Froude number. It is meaningful that the beach slope varies with the inverse of the densimetric Froude number of the discharge jet, because this number is associated with the strength of the jet flow while the resulting slope is a consequence of the loss of energy of the jet that causes material to be deposited on the beach.

The parameter P' is a scaling factor that permits one to normalize tests performed at different scales such as laboratory flume tests and field tests. When plotting the slopes obtained from the flume tests using TS sand (Chapter 3) and the slopes of field tests with the same sand (Chapter 5) in terms of the parameter P', the results define a single curve as shown in Figure 7.19. A single curve of slope versus parameter P' is also obtained when comparing the field and flume tests performed by Boldt (1988) on the same material (Figure 7.20). Unfortunately, it is not possible to evaluate the results presented by the Delft group (de Groot et al., 1988 and Winterwerp et al., 1990) since the parameters associated with the discharge jet that are involved in the parameter P' are not available.

It is striking to observe that both sets of data define the same curve of slope versus P', even though the materials being deposited were so different (Figure 7.21). The tests described in Chapters 3 and 5 utilized a fine sand with D_{50} of the order of 0.18 mm and percentage of fines varying between 0 and 25%, while Boldt (1988) used a material that had a mean grain size one order of magnitude smaller ($D_{50} = 0.014$ mm) and a much larger percentage of fines (≈ 90%).

The scaling of results obtained for similar tests utilizing different materials can also be observed when comparing the results of TS, KS and SS sands (Chapter 3) in terms of the parameter P' (Figure 7.22).

The parameter P' also permits one to normalize other results presented in the literature, including flume and field measurements. When plotting the slopes obtained on the flume tests reviewed in Chapter 4 against the parameter P' calculated for the respective tests, the results tend to be closer to a single curve. Figure 7.23 includes the results of Porto Primavera (UPP) (Ferreira et al., 1980) and USBM tailings B (Boldt, 1988) with the other results already discussed (Figs. 7.19 to 7.22). The last point of Porto Primavera ($P' \approx 3.5$) corresponds to the lowest flow rate for which the pipe possibly was not flowing with full section. The actual value of P' for this point was then probably much lower than 3.5. Although the values for USBM tailings A compared well with the remaining results, the values of P' obtained for tailings B correspond to slopes 1 to 2% higher than the measured slopes. A possible explanation could be the fact that tailings B has a much higher coefficient of uniformity ($C_U = 22$) than any other material analyzed ($C_U = 2$ to 8). Both tailings A and B are very fine and contain a significant percentage of material passing the #200 sieve (tailings A $\approx$ 90%; tailings B $\approx$ 45%), which could have been expected to change the depositional characteristics in relation to the other more granular soils.

Figure 7.24 summarizes the results discussed and shows that these results are described by the following empirical relationship:

$$i_{av} = 5\sqrt{P'} \tag{7.18}$$

It is interesting to observe that the three last points for which the above relationship does not apply well (Figure 7.24) correspond exactly to the three tests that developed non-segregating slurry behaviour. The relatively high slurry concentrations of these flume tests (C_w= 40.5% for TS tests and $C_w > 23\%$ for KS tests) for the existing conditions of flow rate and seepage, led to a different mechanism of deposition with the formation of sand lobes discussed in Chapter 3.

The parameter P' needs to be calculated using the actual values of flow rate, slurry concentration and grain size that promoted the formation of the beach. The use of nominal or average values can lead to relatively large errors in the determination of P'. Taken as an example the field tests described in Chapter 5, the use of the slurry concentration in the pipeline instead of the actual slurry concentration would have caused errors of the order of

5 to 45% in the value of *P'*. The use of the nominal slurry concentration would have caused errors up to 60%.

7.5 - SUMMARY AND CONCLUSIONS

Although the estimate of beach geometry is an important aspect of the design of a hydraulic fill, there are no adequate methods available.

The most promising type of approach seems to be one based on the physics of sediment deposition and that incorporate the knowledge of fluid mechanics and sediment transport. However, both methods that follow this approach (Winterwerp et al., 1990; Fan and Masliyah, 1990) do not compare well with observed values in all cases.

Melent'ev's dimensionless equation provides a reasonable description of the beach shape, but there are still difficulties in determining the exponent *n* at the design stage, as the factors that affect its value are not well understood yet. The application of Melent'ev's equation to predict field profiles requires not only the exponent *n*, but also the overall slope *H*/*L* that cannot be obtained directly from flume tests.

A new parameter is proposed in this paper that permits one to determine the field overall slope from flume tests results. For 6 flume tests and 2 field cases analyzed here, the parameter P' also made it possible to normalize results from tests with very distinct materials deposited under different conditions. A very simple relationship (Eq. 7.18) provides a reasonable estimate of overall slopes as a function of the discharge parameters for all cases of segregating slurries. The main advantage of this parameter is that it relates the resulting slope to outlet parameters over which the designer has total control. However, since it is an empirical parameter, it should be used carefully and within the range of values for which it has been determined. Limitations to the applicability of this parameter will become clearer as more data is collected and analyzed.

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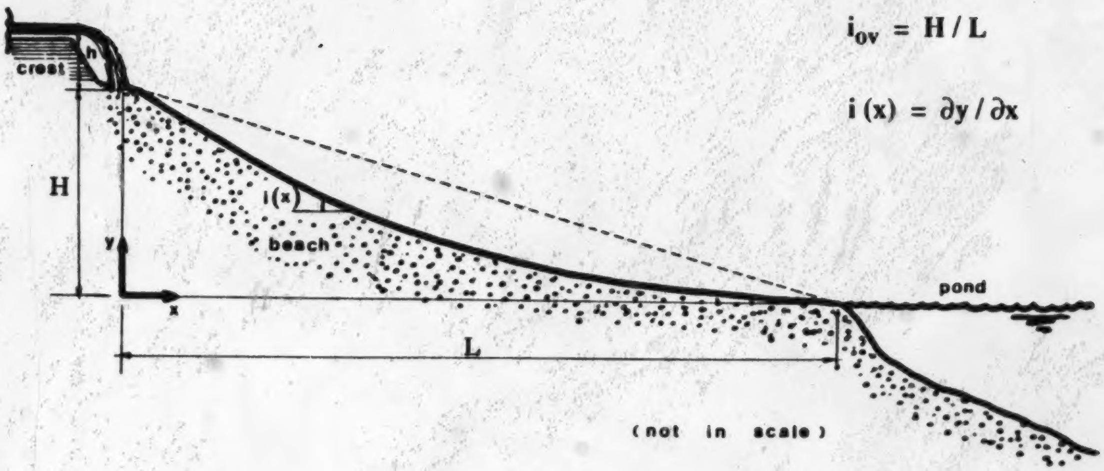
$i_{ov} = H / L$
$i(x) = \partial y / \partial x$
crest
h
H
y
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beach
pond
L
(not in scale)

FIG.7.1 - Sketch of the geometry of a hydraulic fill beach

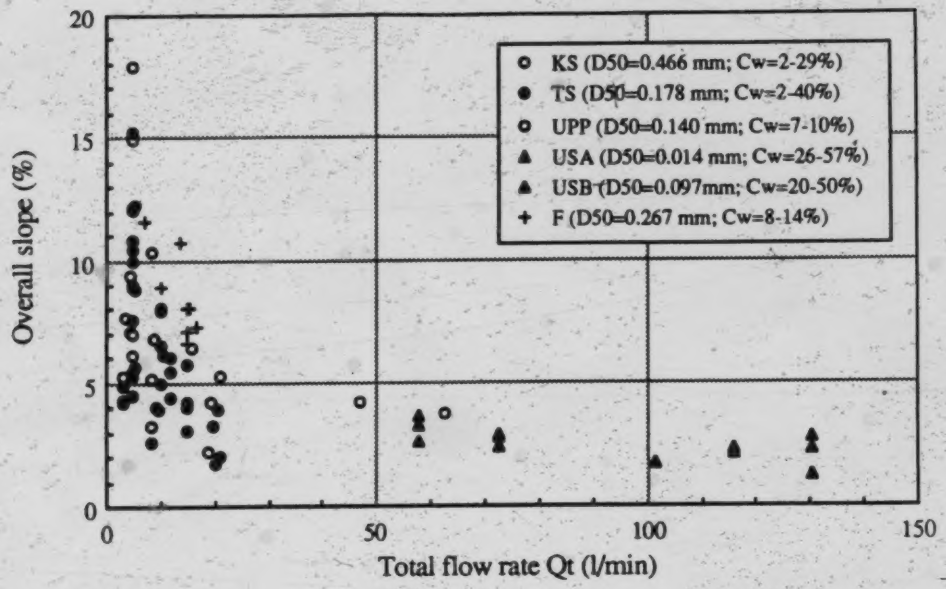

FIG. 7.2 - Effect of flow rate on beach slopes obtained in flume tests

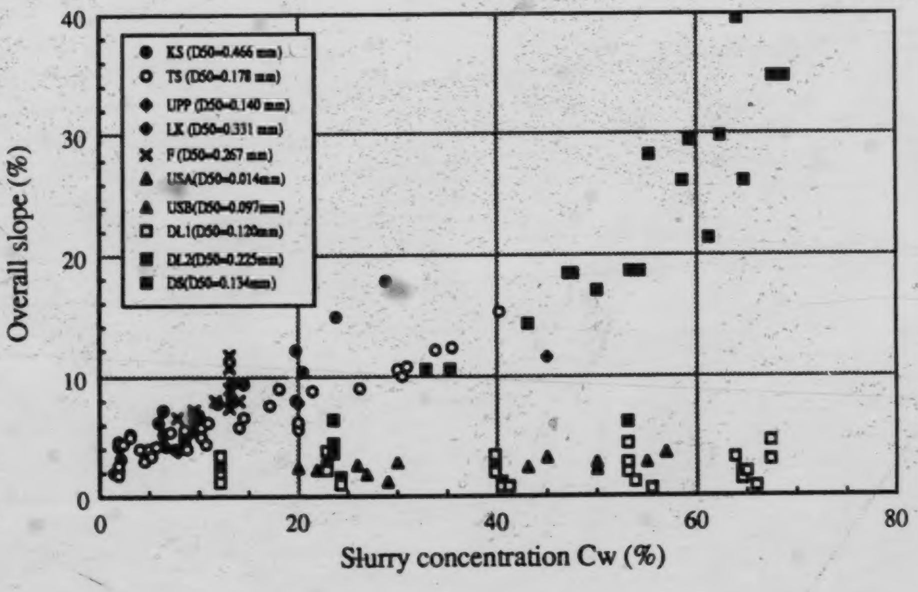

FIG. 7.3 - Effect of slurry concentration on beach slopes obtained in flume tests

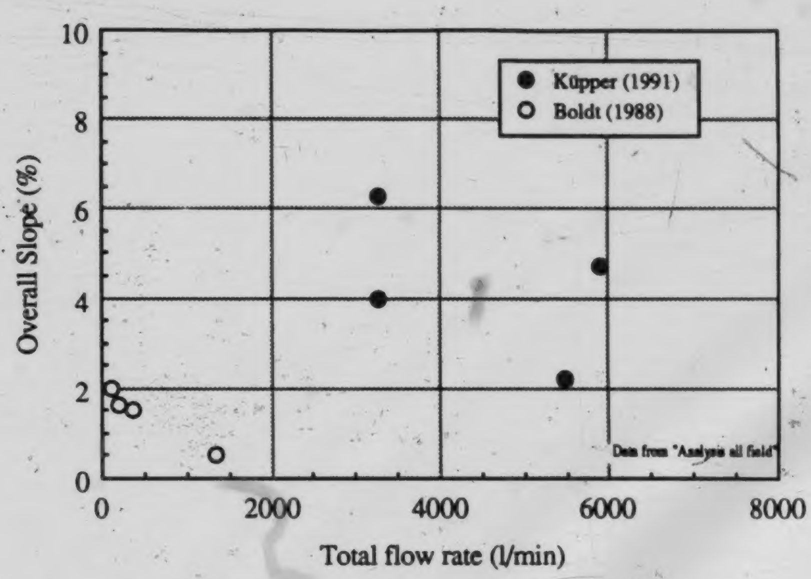

FIG. 7.4 - Effect of flow rate on beach slopes obtained in field tests

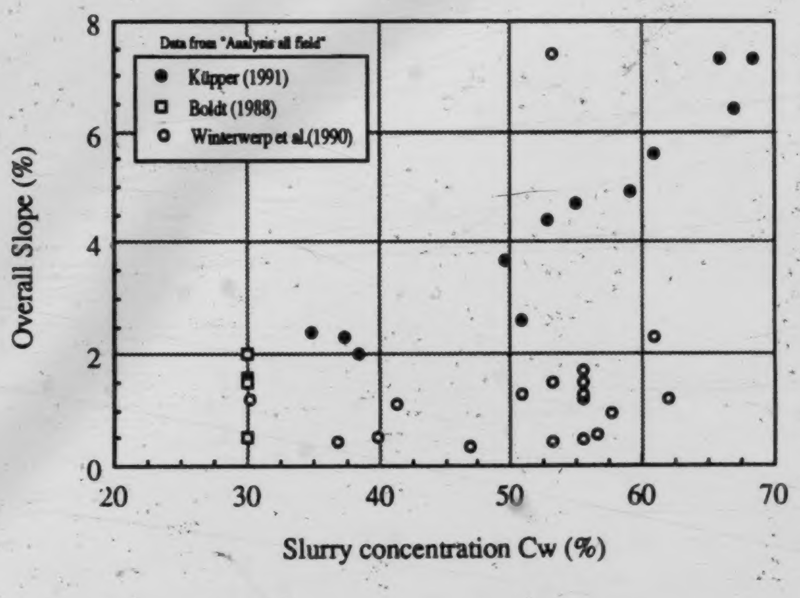

FIG. 7.5 - Effect of slurry concentration on beach slopes obtained in field tests

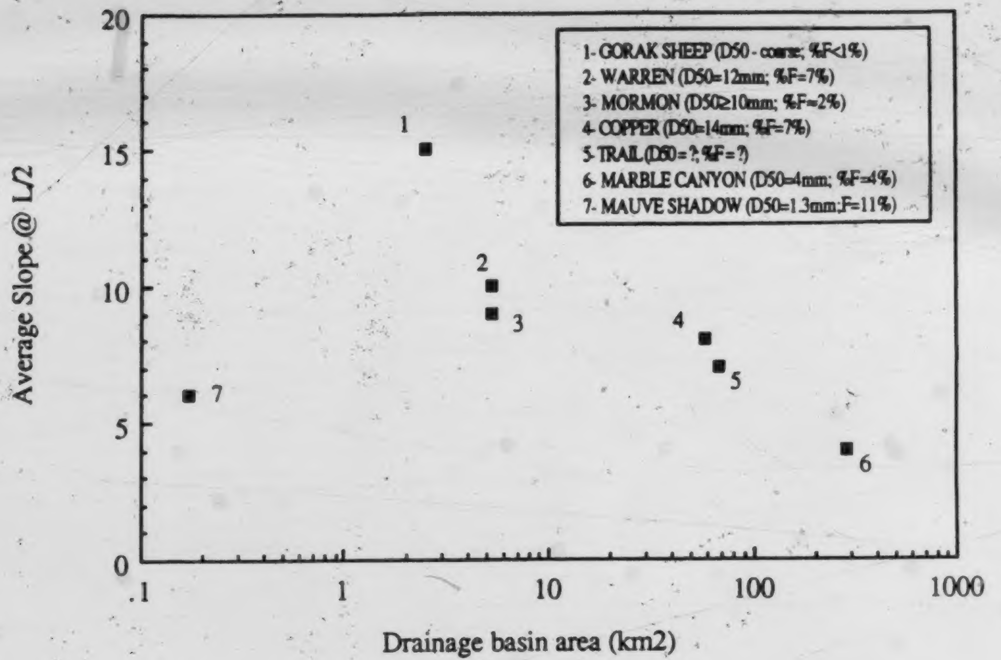

FIG. 7.6 - Variation of alluvial fan slopes with basin area (or flow rate)

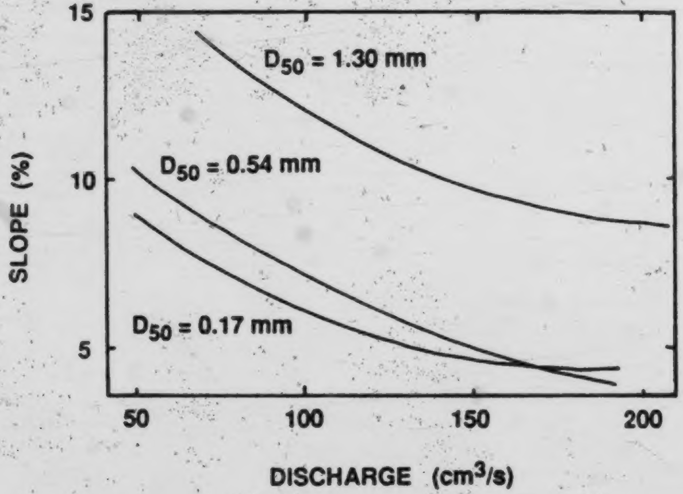

FIG. 7.7 - Relation between slope and discharge for three different sediment sizes (modified after Hooke and Rohrer, 1979)

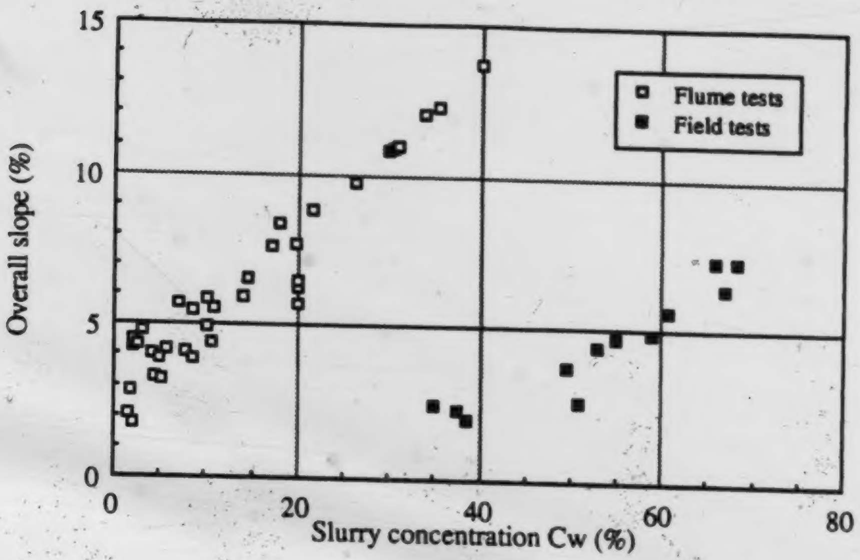

FIG. 7.8 - Comparison between flume and field tests slopes for TS sand

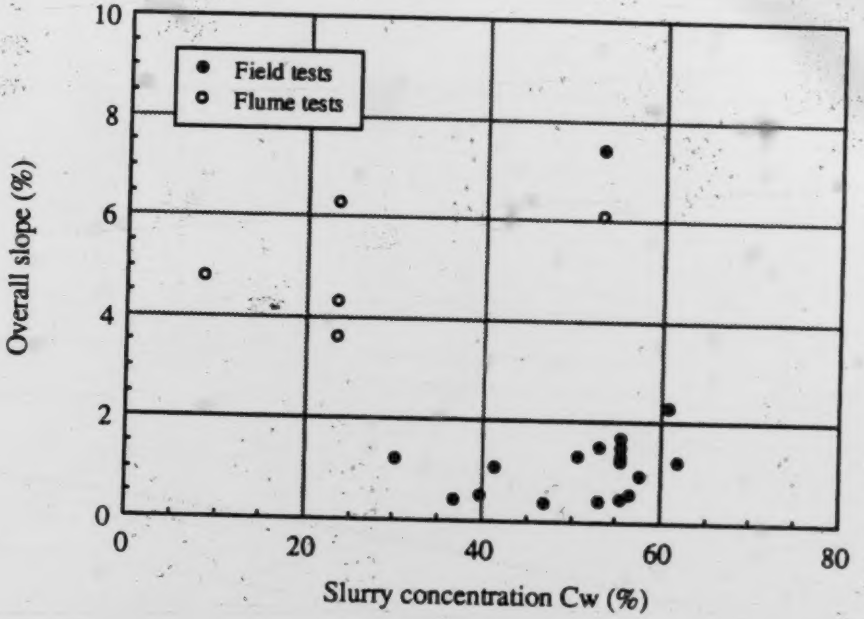

FIG. 7.9 - Comparison between flume and field slopes (Winterwerp et al., 1990)

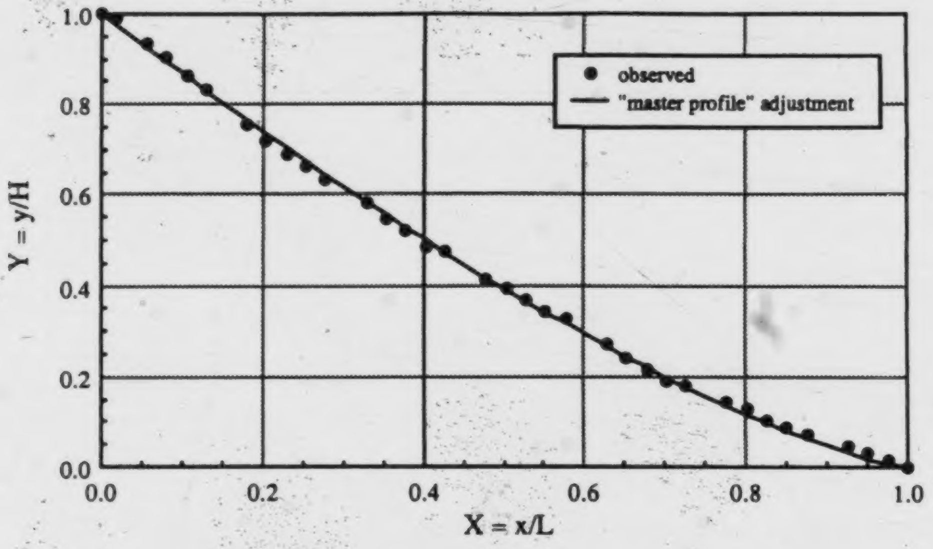

FIG. 7.10 - Master profile adjustment for flume tests TS31

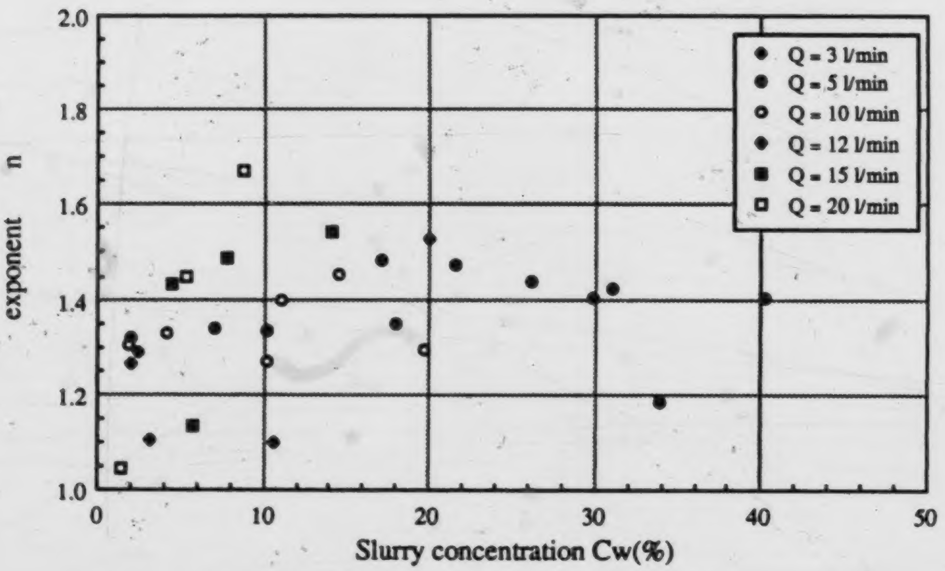

FIG. 7.11 - Master profile exponent n as a function of slurry concentration and flow rate for TS flume tests

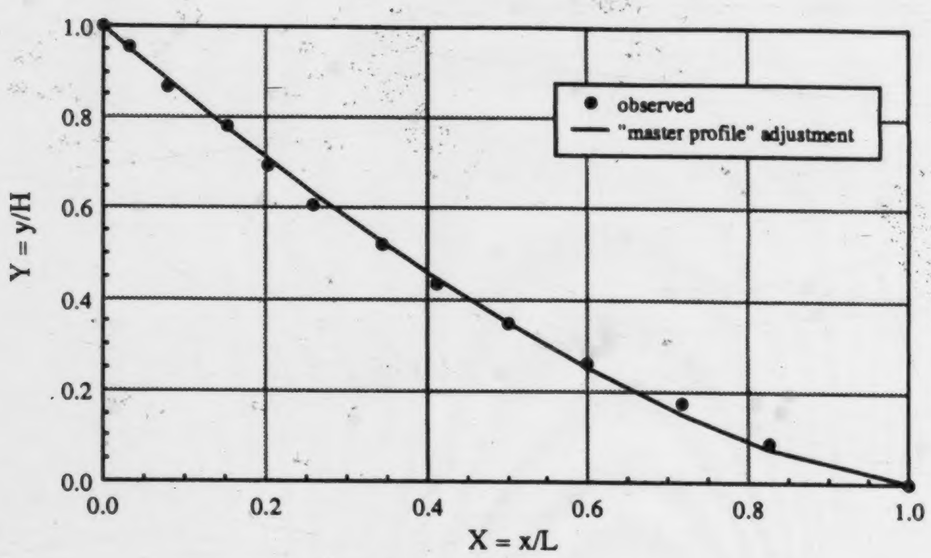

FIG. 7.12 - Master profile adjustment for field tests FT1, Profile III

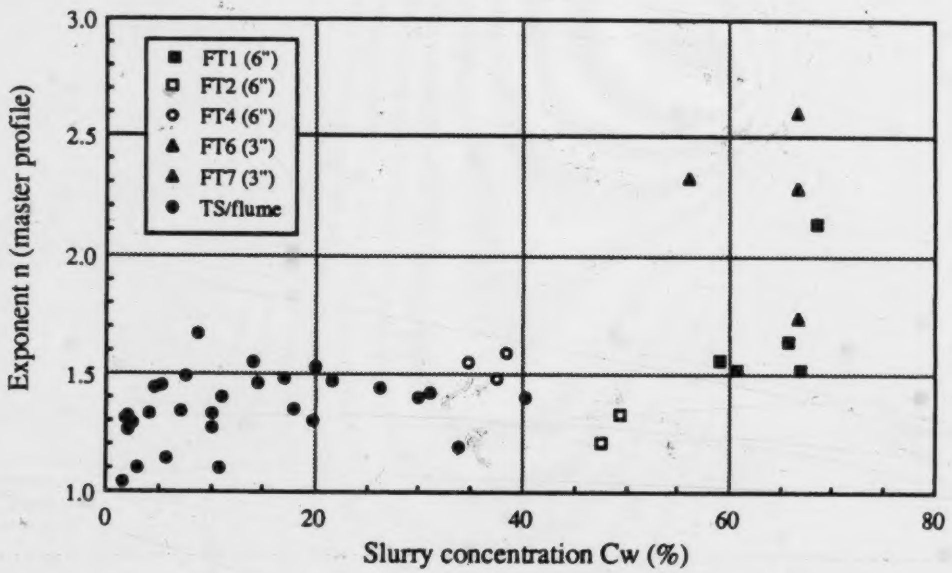

FIG. 7.13 - Master profile exponent n for field (FT) and flume (TS) tests

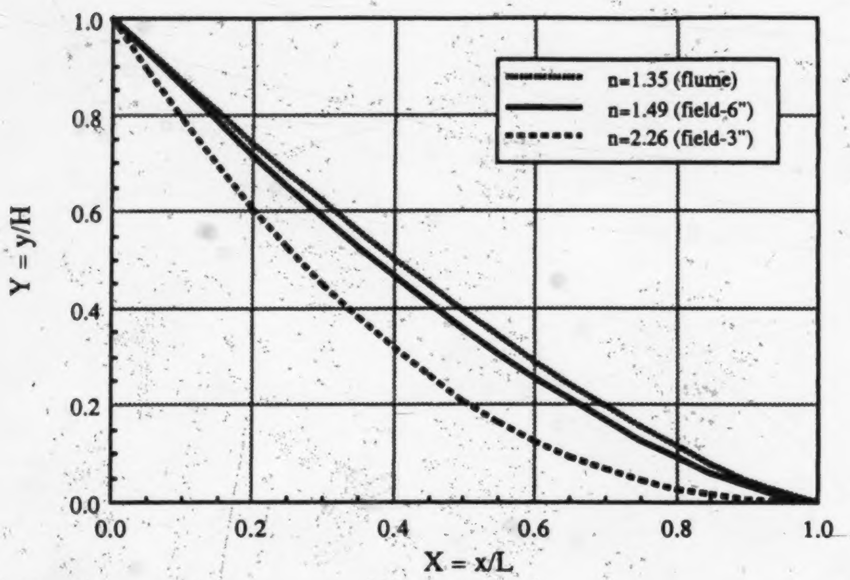

FIG. 7.14 - Master profiles defined by the values of n obtained from flume and field tests

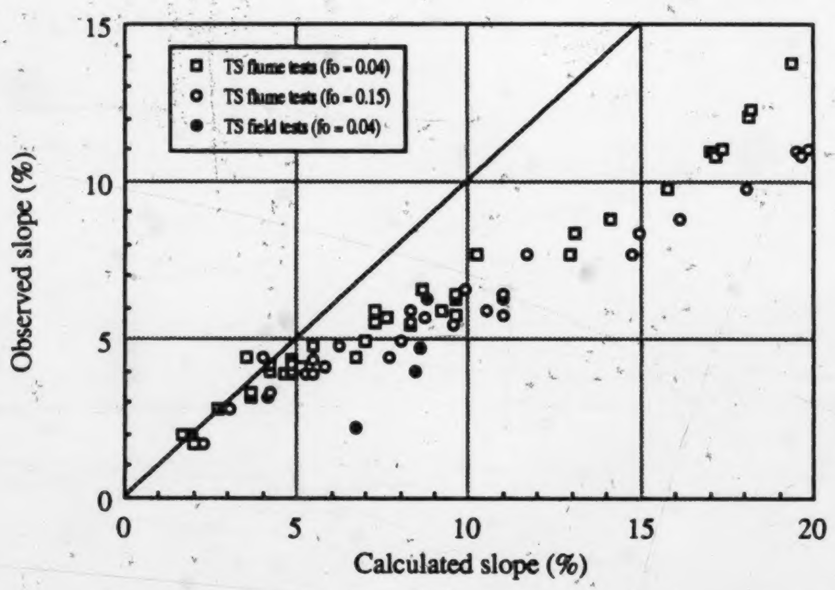

FIG. 7.15 - Evaluation of equation 7.14 (see text) to predict beach slopes

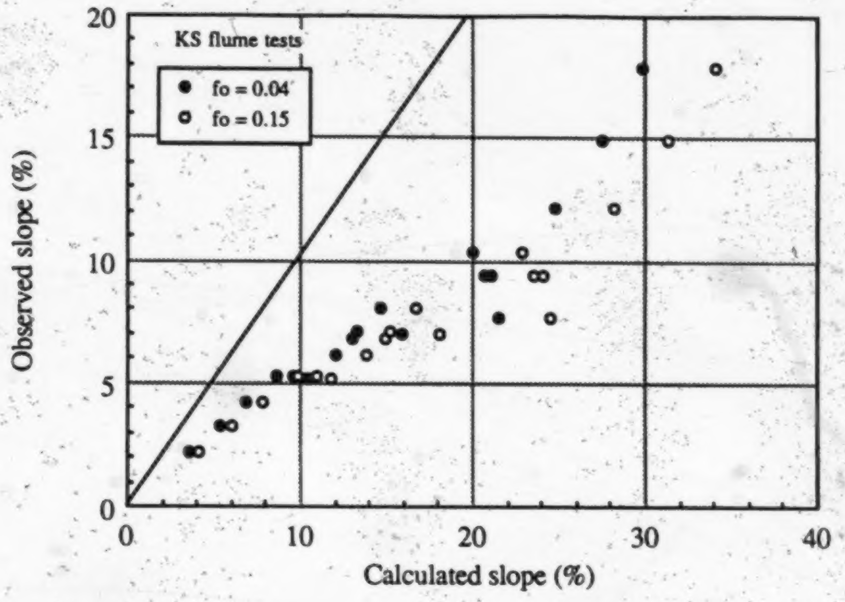

FIG. 7.16 - Evaluation of equation (7.14) (see text) to predict beach slopes

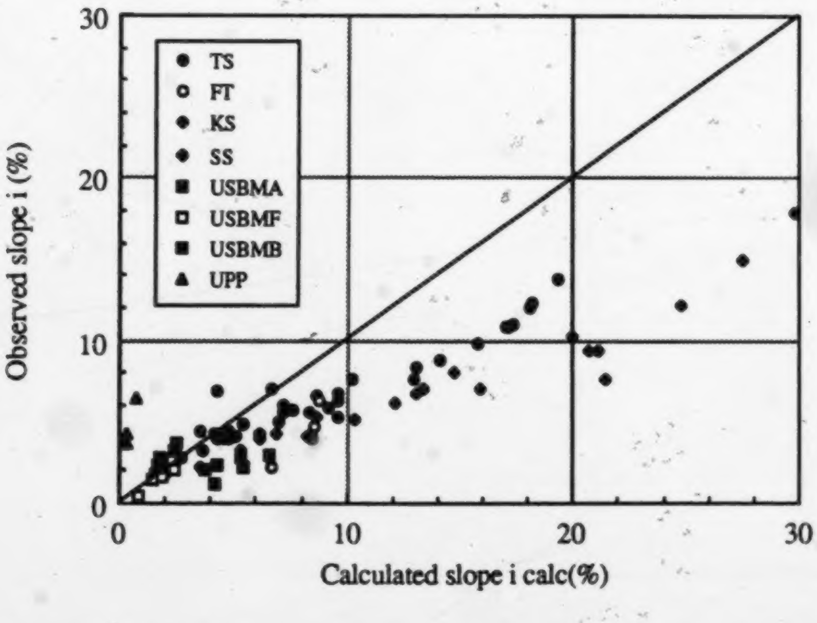

FIG. 7.17 - Slopes calculated using Dutch model (Winterwerp et al.,1990) for fo = 0.04

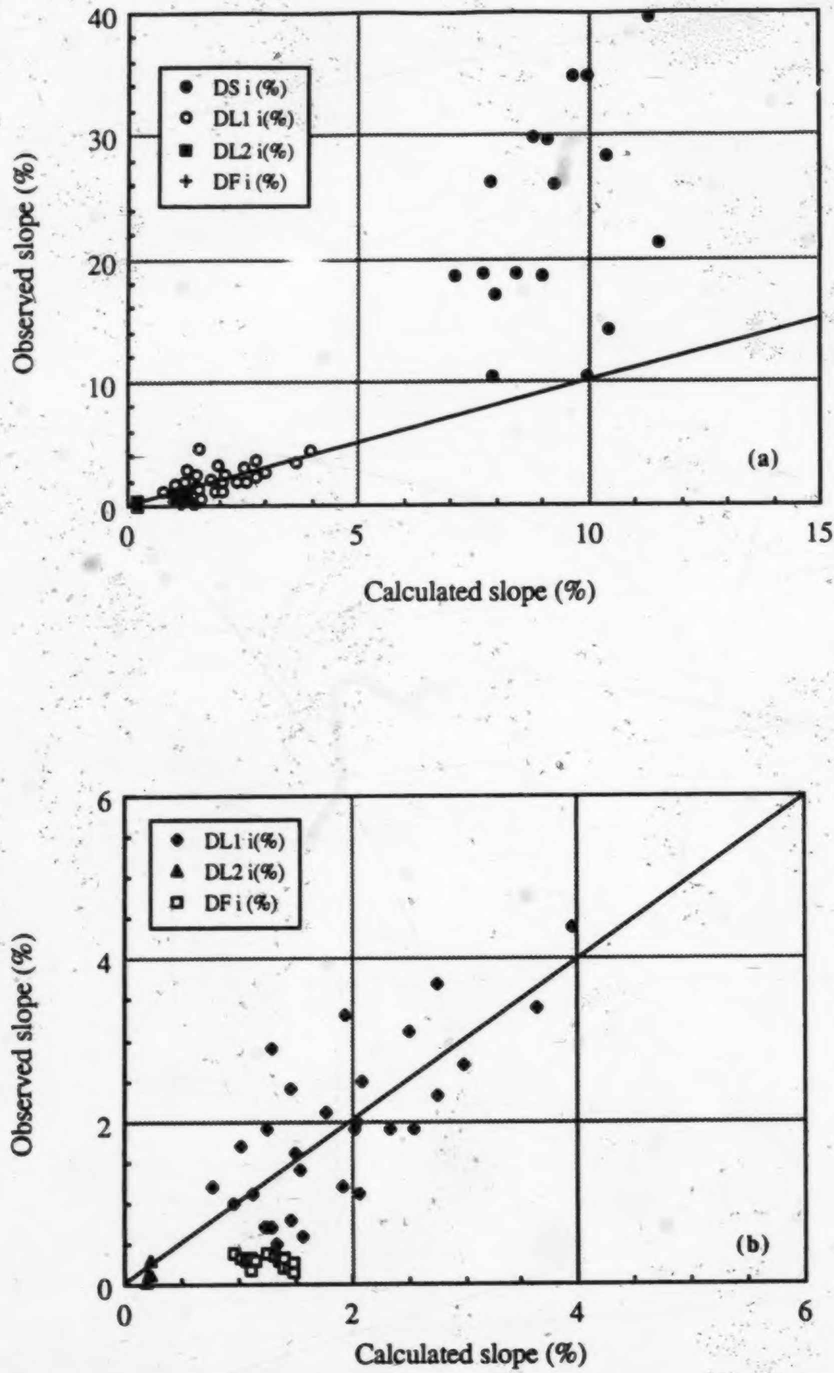

FIG. 7.18 - Evaluation of equation (7.14) to predict results of tests from Winterwerp et al.(1990)

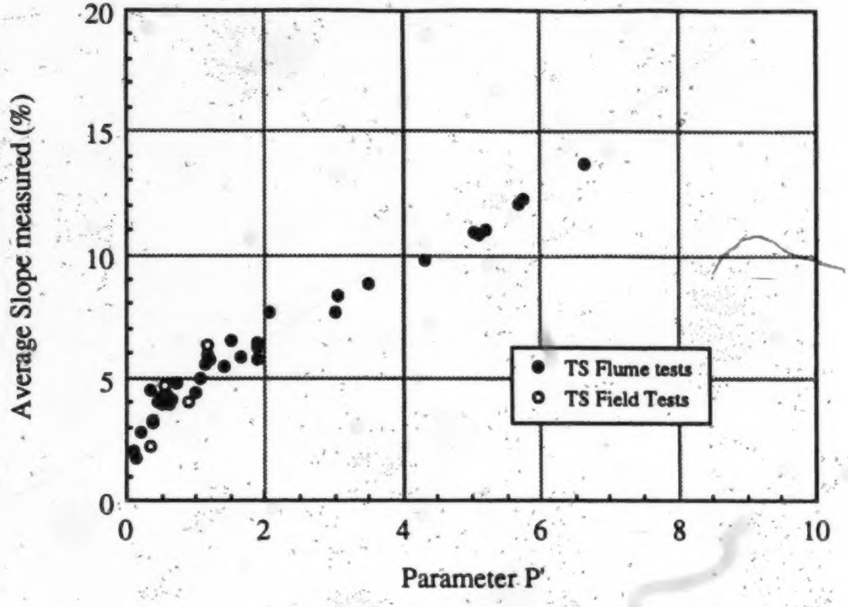

FIG. 7.19 - Normalization of field and flume results using the parameter P'

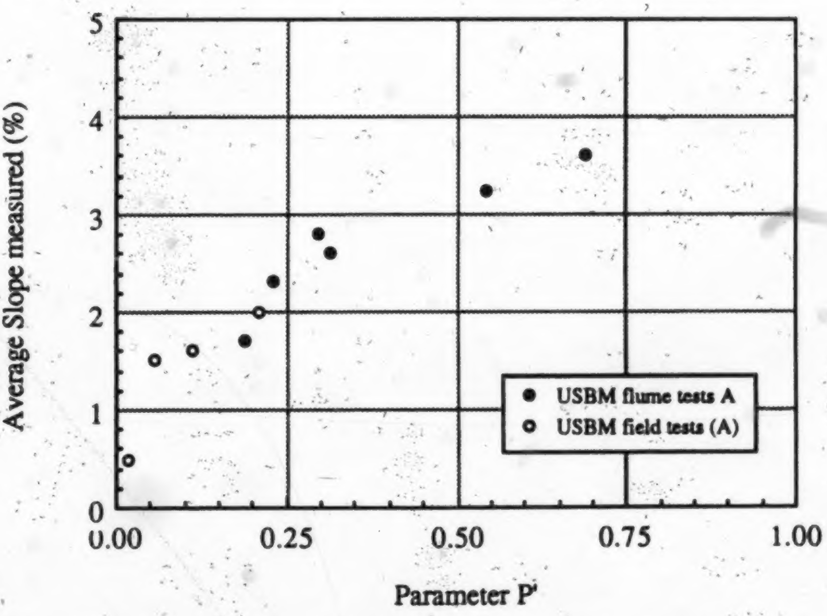

FIG. 7.20 - Normalization of the flume and field results obtained by Boldt (1988)

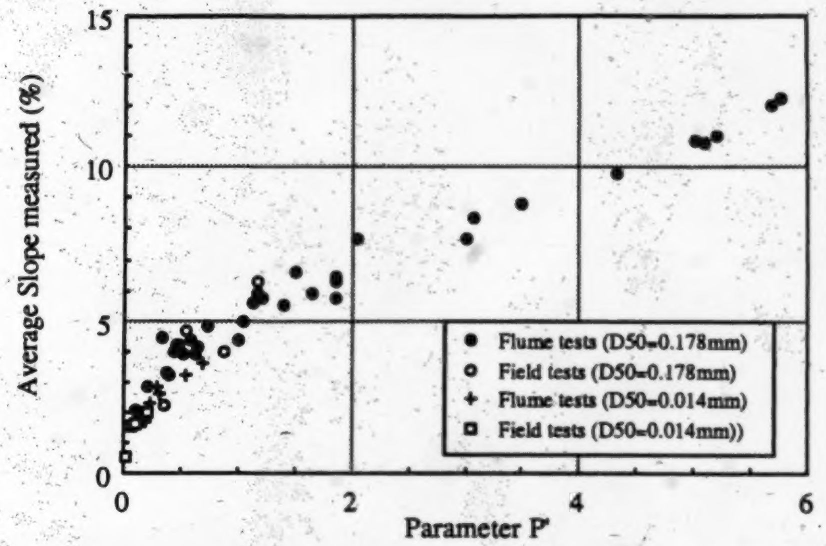

FIG. 7.21 - Normalization of flume and field results for different materials: D50=0.178mm (TS sand) and D50=0.014mm (Boldt,1988)

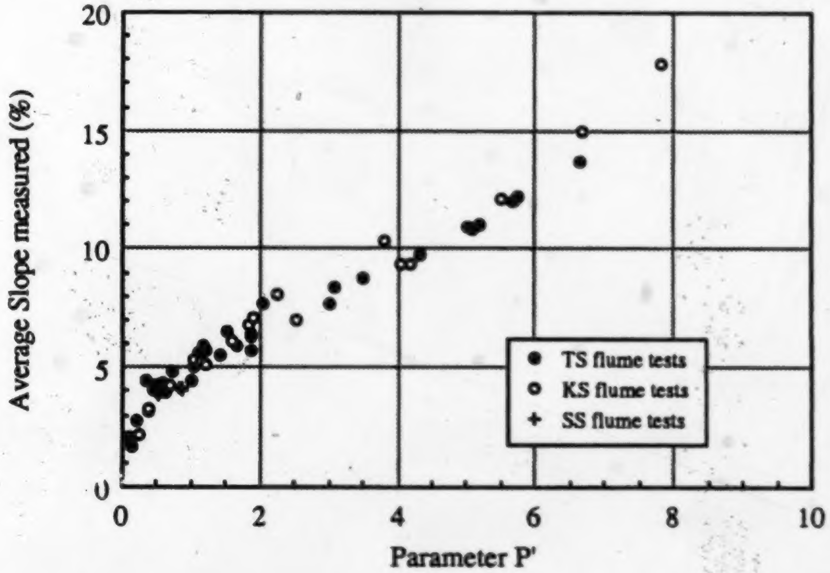

FIG. 7.22 - Normalization of slopes for flume tests using different materials

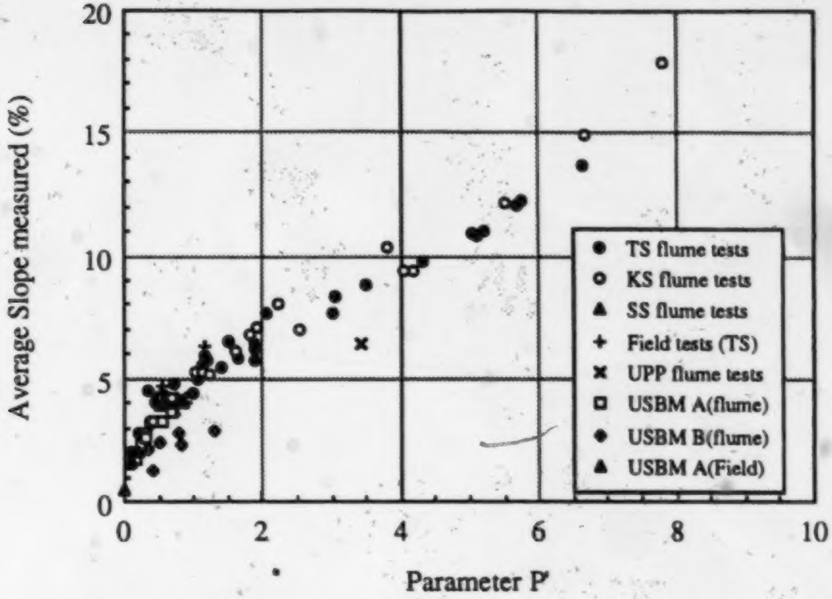

FIG. 7.23 - Normalization of flume and field tests presented in the literature

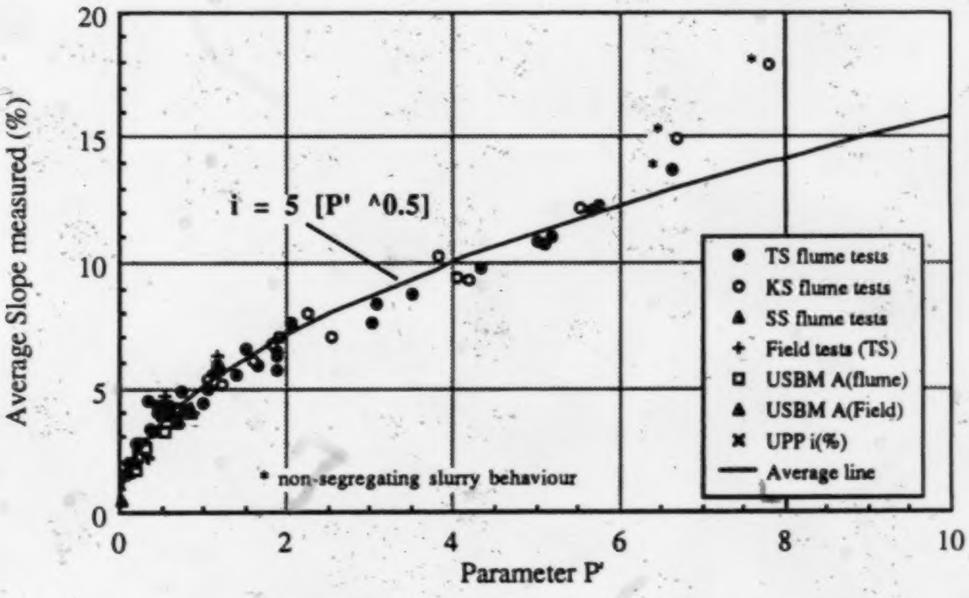

FIG. 7.24 - Normalization of beach slope using the parameter P'

Chapter 8

Density of Hydraulic Fills

8.1 - Introduction

There are several important points that should be considered in the design of hydraulic fills, and a particularly critical one is the density of the fill material. Obtaining a relatively high density is essential for the stability of the structure under both static and dynamic conditions. Hydraulic fills are particularly susceptible to liquefaction because the deposition conditions favour the formation of clean uniform saturated sand deposits. Thus, the density of the fill becomes especially important for hydraulic fills. For waste disposal structures such as tailings dams and dredging disposal sites, a higher density of the deposited material has the additional benefit of increasing the lifetime of the facility.

Given the importance of the density on the performance of a hydraulic fill, it is of interest to have a method to design the fill in a way to maximize its density.

For the design of compacted fills, the density can be estimated at the design stage. It is known that for a given material the density increases as the compaction energy increases and as the water content approaches the optimum value. These parameters are evaluated in the laboratory at the design stage, so that the construction method can be specified to obtain a fill with an adequate density.

There is nothing similar for hydraulic fills, except for empirical recommendations found in the Soviet literature, but that have not been adopted in the Occidental World. The Soviet recommend to maximize flow rate and minimize discharge velocity. They also use relatively low slurry concentrations and low rates of filling to achieve higher densities.

Therefore there is a need for a method of designing hydraulic fills to maximize densities. Even if the exact value of density cannot be known at the design stage, it would be of interest to have a basis for specifying a construction method that produces a relatively high density, which if not high enough to assure stability, at least would be as high as possible in order to reduce the densification costs.

This chapter explores qualitatively the issue of density of hydraulic fills from the viewpoint of the physics of the hydraulic deposition process. This process has already being studied in other disciplines such as fluvial hydraulics, sediment transport and sedimentology. These various fields have diverse objectives and as such, use different approaches, different range of values for the parameters involved and also use different terminology. Hydrodynamic studies tend to focus on the details of the flow and its interaction with the sediment in motion and with the top layer of the deposited (stationary) sediment. Sedimentologists are mainly interested on the description of the resulting sedimentary structures. Over the last two or three decades, sedimentologic studies started considering the flow conditions and lately more detailed quantitative analysis have been incorporated. These interests are different from the interests of geotechnical engineering, which is more concerned with the physico-mechanical properties of the material being deposited. The hydraulics of the flow on the hydraulic fill beach is never an issue of detailed consideration.

The main objective of this chapter is to review some concepts developed in other disciplines and indicate how they could provide useful guidelines for the control of densities in hydraulic fills. The concepts of bed forms and of flow regime are examined and a possible philosophy for the analysis of hydraulic fills is described. Due to the interdisciplinary character of the analysis of hydraulic deposition, some basic concepts are briefly presented since they may not be totally familiar to professionals from other areas.

8.2 - Some physics of hydraulic deposition

This section describes some physical aspects of the hydraulic deposition process that may have a bearing on the density of the resulting deposit.

The construction of a hydraulic fill consists basically of discharging a mixture of solids and fluid onto an area, where most of the solid is deposited. In segregating slurries the water and the grains will behave as separate phases, as opposed to non-segregating slurries that behave as a mono-phasic viscous fluids. A segregating slurry is used in most cases of hydraulic fills. After the slurry is discharged, the grains tend either to deposit or to flow close to the surface of the deposit, constituting what is called bed load. In this case, the segregation process creates a situation of a flow of a fluid over an erodible boundary.

As a fluid flows on a surface that has erodible boundaries, an interaction is established between the fluid and the boundary material. The flow will change the boundary by eroding and depositing material and thus changing the configuration of the bed surface. The sediment moves and gets organized into morphological elements called bed forms. The movable boundary will also affect the flow conditions by deforming the flow lines and by imposing resistance to flow. A complex interaction is developed between the coherent turbulent structures in the flow and the geometry and physical properties of the bed.

Since fluid flow over erodible boundaries is the physical phenomenon that underlies deposition processes on a hydraulic fill beach, the understanding of the interaction between flow dynamics and the properties of the bed is essential for the rational design of hydraulic fills. The density of the fill seems to be particularly affected by how the fluid and the boundary interact, therefore this issue will be discussed in some detail in the next section.

8.2.1 - The concept and the organization of bed forms

Flow of a fluid over a rough boundary applies shear stress to this boundary. When the boundary shear stress exerted by the fluid on a flat sand bed exceeds a certain critical value, sand grains will begin to move.

After this stage a small increase in flow velocity causes movement of grains in such a way that the bed becomes covered by small asymmetrical wavy forms called ripples (Figure 8.1). Ripples are controlled by flow conditions in the viscous boundary layer (Williams and Kemp, 1971; Yalin, 1977) and therefore are independent of the flow depth, as verified experimentally by Allen (1963). Flow over ripples presents a pattern of flow separation at the crest and flow reattachment downstream of the trough (Figure 8.1). Grains are moved, as bedload, up the ripple stoss side until they fall or diffuse from the separating flow at the crest on to the steep ripple lee face. Small grain avalanches and sedimentation of grains that were suspended at the crest deposit a lamina on the ripple lee (Jopling, 1964; Leeder, 1982). This lee accretion causes dislocation of the flow reattachment point up the back of the downstream ripple, where increased erosion occurs because of the high turbulent stresses generated at the reattachment point. In this way the ripples constantly shift downstream, preserving their overall equilibrium shapes. A vertical section parallel to the flow shows that a ripple deposit presents small scale cross-stratification, but if the rate of net sediment deposition is high, it may develop a wavy lamination (Allen, 1972).

Further increase in flow velocity will cause these ripples to grow into larger wavy forms called dunes and will cause the sediment transport rate to increase. Dunes are similar to ripples in their general shape (Figure 8.1), but are dominated by processes acting on the whole boundary layer rather than just on the viscous sublayer (Jackson, 1976; Yalin, 1977). Consequently, dune characteristics depend on the flow depth. The flow pattern over dunes is similar to that over ripples, with well developed flow separation and reattachment. Large scale cross-stratification is the characteristic sedimentary structure formed under dune conditions.

If the flow velocity increases even more, the dunes will be gradually wiped out, and after a transition stage, the bed will become flat (Simons and Richardson, 1960, 1961). The flow regime that occurs before this transition stage is called lower flow regime and is characterized by relatively high flow resistance, small sediment transport rate and subcritical flow. After the transition stage, the flow resistance decreases, the sediment transport rate increases and the flow tends to be supercritical. It is called upper flow regime (Simons et al., 1965).

The plane bed that follows the transition stage (Figure 8.1) presents a low resistance to flow, which results mainly from grain roughness. It is associated with intense sediment transport (Harms and Fahnestock, 1965) with most of the transported material being confined to a thin layer close to the bed (bed load). Sand moves continuously on the surface without much evidence of large scale turbulence. The pattern of the coherent turbulent structures of the viscous sublayer causes the bed surface to be marked by a system of low linear ridges, a few grain diameters high and aligned parallel to the flow direction, called primary current lineations (Stokes, 1947, p.42). The presence of current lineation constitutes an excellent evidence for upper regime flows (Allen, 1963, 1964) and of the tractional character of flows with significant bed load transport (Harms et al., 1982).

Upper stage plane bed deposits have internal structure of planar, almost horizontal laminations ranging between 5 and 20 grains thick (Leeder, 1982) and varying slightly in composition and sorting. These types of laminae have been described by many authors (see summary in Bridge, 1978). The mechanism of formation of the planar laminae of upper-stage plane beds is commonly accepted as being associated with the burst-and-sweep structures of the turbulent boundary layer (Bridge, 1978). However, the details of the mechanism are still being debated (Cheel, 1984, 1990a, 1990b; Cheel and Middleton, 1986a, 1986b; Allen, 1984; Bridge and Best, 1988; Paola et al., 1989). Details of the origin and characteristics of lamination and primary current lineation are important because both features result from the more subtle flow / bed interaction that occurs under upper-stage plane bed regimes and their study helps understanding this interaction.

After upper-stage plane beds are established, an increase in velocity causes waves on the water surface that are in phase with sand waves on the bed and are called antidunes (Figure 8.1). Antidunes result from the interaction between the free-surface and the bed (Ismail, 1952; Hill et al., 1969; Harms et al.1982). Antidunes commonly occur as long trains of symmetrical waves in very fast shallow flows with Froude number larger than 0.7. The antidune stage of flow comprises a range of energy levels. At relatively low energy levels, small waves are formed in-phase with the sand waves. The flow resistance is similar to the flow resistance for plane beds, the sediment transport rate is slightly higher (Simons et al., 1965) and the waves migrate downstream. At increased energy levels, the waves tend to remain stationary and are commonly called "standing waves". At higher energy levels, the water waves gradually steepen, move upstream and eventually break, with the process beginning over again on a cycle associated with growth and partial destruction of the bedforms. Prior to wave breaking, a flow separation zone forms upstream of the crest. Sedimentary structures formed by antidune flows are characterized by low angle faint laminations.

As antidune flows are further increased in velocity, chute-and-pool structures are formed (Figure 8.1). This situation is marked by sequences of steep chutes where a shallow supercritical flow accelerates before entering abruptly in the deeper subcritical pool, forming a hydraulic jump. Sediment accumulation may occur in the relatively tranquil pool region where steeply dipping backset laminations may develop (Leeder, 1982). The internal structures of chute-and-pool deposits are poorly understood in comparison with other types of deposits. Chutes and pools are usually associated with steep overall slopes and high sediment discharges.

This sequence of different bed configurations with increasing flow strength was first systematized by Simons and Richardson (1961) based on an extensive flume test program. Results of these classical experiments were summarized by Guy et al. (1966).

Bed forms have been observed in natural and laboratory flows under a variety of flow conditions. They depend on the nature of the flow and the size of the sediment and in each case, each kind of bed form is stable only between certain values of flow strength. Bed form migration can occur under conditions of net deposition of sediments, equilibrium or net erosion (Langford and Bracken, 1987).

8.2.2 - Bed forms associated with hydraulic fills

As discussed above, granular materials deposited under different flow conditions will develop distinct sedimentary structures. Moreover, they will have different fabric and structure, and consequently different geotechnical behaviour. The behaviour of the deposits corresponding to each bed configuration has not been studied as such. However, by analyzing the mechanics of deposition in each case, it might be possible to reason what to expect from the behaviour of the sedimentary structures associated with each mode of deposition.

Due to the relatively high energy level imposed by a typical slurry discharge and by the high sediment concentration of the slurry, ripples and dunes do not occur under hydraulic fill conditions, except in localized areas of the low part of the beach. Generally under these conditions, the flow regime is on the upper stage with bed configurations being plane beds, antidunes or chutes-and-pools, as observed in several hydraulic fills. However, since dunes are the lower limit for upper stage plane beds, they will be included in this discussion.

The mechanics of sediment movement on a dune is associated with the pattern of flow separation at the dune crest and flow reattachment at the stoss side of the next dune downstream. When the flow separates at the crest, all grains that had been carried out as bed load over the stoss side are put in suspension (Figure 8.2a). Most of this material is too heavy to be in suspension for a long time and gets rained over the lee side and the trough. It is well known that granular materials that are rained into water have a lower density than materials formed by other deposition methods (Kolbuszewski, 1950Vasques, 1990; Chapter 3) and certainly a lower density than materials deposited by flowing water. The reverse flow in the trough may even carry sediment up the lee face where it is deposited in such a way that the stability is only guaranteed by the reverse flow forces acting against gravity. Simons et al.(1965) report lee slopes angles of 4 degrees above the angle of repose. Material deposited under these conditions will be clearly in a very loose state. On the stoss side, however, the flow is parallel to the bed, it is accelerating and the bed shear stresses are relatively high. Therefore, if a grain is not well placed and well imbricated, it will be carried away either to be deposited further downstream in a well locked position on the stoss side or to be suspended at the dune crest. Thus, it is reasonable to expect the material on the stoss side to be relatively firm. The lee side grows downstream and the stoss side moves also downstream over lee-deposited material (Figure 8.2a). As the dominant deposition occurs on the lee side, the overall density of the resulting deposit is likely to be low. In any case, the overall behaviour of the deposit will probably be unstable since thin firmer layers alternate with thicker soft layers (Figure 8.2b).

The dynamics of flow and sediment transport over upper stage plane beds is very similar to what happens on the stoss sides of dunes. The bed shear stress is high and grains that are not placed in a well imbricated tight position will be carried away. Therefore, whatever grains remain deposited are likely to form a relatively dense deposit, provided the rate of deposition is not so high that grains get buried before finding a stable position in the bed. This is actually the basic principle that relates higher densities to lower rates of deposition or lower slurry concentrations (Kolbuszweski, 1950; Gray, 1968; Lieng et al., 1985). The flow over plane beds does not present separation or large scale turbulence that could lift grains up and disturb the deposition process or cause grains to be rained over the bed. Consequently a relatively high density is expected for materials deposited under these conditions. Also, deposits formed under upper-stage plane bed flows are characterized by thin horizontal (or near horizontal) planar laminae that can have a significant influence on the material behaviour. These deposits are likely to have an adequate geotechnical behaviour.

The dynamics of low energy level antidunes is very similar to upper stage plane beds, except for the waviness of the bed and of the water surface. In neither of these cases is there flow separation or any major disturbance of the bed by large scale turbulence and the bed shear stress is high in both situations. The deposit formed under low energy antidunes is then not expected to be very different from plane bed deposits in terms of geotechnical behaviour, and thus may be satisfactory for hydraulic fills. For higher energy level antidunes, there is flow separation just before the water wave breaks on a zone upstream from the crest. Although relatively weak, this separation zone is probably enough to cause a loosening of the deposit. When the energy level is high enough, waves break throwing in suspension large amounts of sediment and disturbing the bed. After the wave turbulence settles, the suspended sediment is rained on the bed, becoming deposited in a loose state. Simons et al. (1965) report that with the breaking of waves "the bed was disturbed to a considerable depth". Based on several experimental and field observations (Kennedy, 1963; Allen, 1966, 1984; McBride et al., 1975; Barwis and Hayes, 1985; Rust and Gibling, 1990), Cheel (1990) proposes a subdivision of "antidunes" into four subgroups of increasing energy levels according to flow dynamics and bed characteristics (Figure 8. 3). This classification is consistent with the observations of Middleton (1965) and Langford and Bracken (1987). Adopting Cheel's classification, it is reasonable to expect downstream-migrating inphase waves that produce horizontal lamination to be possibly acceptable for hydraulic fills, but not the higher energy level situations.

The flow dynamics of chute-and-pool conditions are also expected to form disturbed deposits that are bound not to have adequate mechanical behaviour. The high energy flow on the chute is erosive and the hydraulic jump at the end of the chute throws into suspension most of the sediment transported on the chute. The subcritical flow in the pool immediately after the hydraulic jump causes the sediment in suspension to be rained down forming a loose deposit (Figure 8.1). In the flume tests performed by Simons et al.(1965), the hydraulic jumps moved upstream at velocities of about 0.3 to 0.6 m/min and the bed was disturbed to a depth approximately equal to the mean flow depth.

Therefore, even though specific geotechnical tests were not performed, it is possible to anticipate that based on sedimentary mechanics, upper-stage plane beds are likely to present the most favourable mechanical behaviour. Possibly, low-energy level antidune deposits would also be acceptable in terms of geotechnical performance.

In fact, the mechanistic reasoning of the expected behaviour of the different deposits is supported by the description of the flume tests carried out by Simons and Richardson at Colorado State University from 1956 to 1961, sponsored by the U.S. Geological Survey. These classical tests provided the basis for the understanding we have nowadays of the dynamics of flows over movable boundaries and of the sedimentary structures created under different flow conditions.

Most ripple and small dune beds are described (Guy et al., 1966) in several of these tests as "very soft and fluid-like" and beds of dunes with ripples superimposed are described as even softer. For larger dunes (with no ripples on the stoss side) the bed is repeatedly reported to be "very firm on the back of the dune" where the flow caused formation of a "rather compact crust, possibly as much as one-half inch thick" and "very soft in the troughs, on the crest and on the avalanche faces" (Guy et al., 1966, p.I22). It was also observed "the top layer on the back of large dunes was firm, but it was very easily broken to expose the softer body of the dune" (op.cit., p.I23). This observation supports the idea that despite the firmer parts, the whole deposit formed by dunes is still very soft. In fact, the film produced by the U.S. Geological Survey (1966) featuring these tests shows the prompt liquefaction of the dunes at a small impact applied to the flume by a technician. At the transition stage between dunes and upper-stage plane beds, "the bed was not soft, but neither was it as firm as during a plane bed run" (op.cit., p.I29). For all plane bed runs the bed surface is described as being "very firm". And for antidunes the "bed was firm in the trough, firm on the downstream side of the waves, softer on the upstream side of the sand wave and much softer on the crest or peak of the sand wave" (op.cit., p.I30). There are also observations on the large amounts of sediment thrown into suspension where the waves were breaking.

It is important to note that these tests were carried out by hydraulic engineers who were not particularly interested on the density of the bed material. However, the difference in density of the bed formed under the various flow conditions was so obvious that it drew their attention (Guy et al., 1966). For these researchers, the softer bed created problems for measuring flow depth as the point gauge would penetrate the bed. On firmer beds, the point gauge would simply stop on the bed. This information on bed densities is likely to be even more reliable than densities measured by sampling because:

a) most sampling methods (if not all) would have introduced disturbance especially when dealing with clean uniform sand and such soft beds as the ripple and dune beds described by Guy et al.(1966).

b) any sample of a reasonable size to allow accurate density measurements would have included several layers and possibly would not have been representative of the local phenomena, as for example sampling the stoss side of a dune.

c) due to the test procedure utilized for these tests, only the upper layers of the deposit were formed under the current flow conditions and therefore it would have been difficult to obtain a representative sample of reasonable size.

The conclusion that upper-stage plane bed deposits are the optimum material for hydraulic fills, low energy level antidunes are possibly acceptable and higher energy level antidunes are not adequate is completely consistent with the description of Run 17 of Simons and Richardson's tests (using a fine uniform sand with $D_{50} = 0.19$ mm):

> "In the first 60-70 ft the flume had a plane bed. Downstream the bed condition became one of small standing waves and then, with an increase in wave size, antidunes in the vicinity of stations 90-100. The most consistent and greatest antidune activity was downstream from station 110. The bed was very firm in the plane bed areas and got softer with increasing wave activity in the downstream direction." (Guy et al.,1966; p.I18)

Yufin (1965, Chapter 28) refers to work carried out by Russinov, in which the structure of hydraulic fills is classified as microstratified, stratified and remolded. Russinov showed that microstratified deposits have the highest density for given conditions, followed by stratified and remolded deposits, the latter having the lowest density (Figure 8.4). According to this graph, the difference in density between microstratified and stratified is larger than the difference between stratified and remolded, accentuating the importance of having the right flow conditions to create a microstratified deposit. However, Yufin does not explain which are these flow conditions and does not give the reference of Russinov's work.

In his studies of alluvial fans in Poland, Rachocki (1981) presents a cross-section of a deposit formed under upper-stage plane bed condition and a cross-section of an antidune deposit. These deposits are shown in Photos 1 and 2, respectively. The difference between the deposits presented in these photos is such that it would be reasonable to consider the upper-stage plane bed deposit in Photo 1 as "microstratified" in comparison with the "stratified" antidune deposit shown in Photo 2. If this is the case, the conclusion reached in the above paragraphs that the upper-stage plane bed is the optimum condition for hydraulic fills would be consistent with the Russian technology, that is known for producing hydraulic fills that perform well, even under seismic conditions.

This conclusion is further supported by observations of Dashi-he Dam in China. This tailings dam survived the Tangsham earthquake (1976, M = 7.8; epicentre at 18 km from the dam site) followed by a magnitude 7.1 aftershock some hours later with minor cracks on the upstream shell and some fissures and sand boils close to the water line. The flow conditions during beach deposition on this dam featured upper-stage plane bed close to the discharge point and some low-energy antidunes further downstream (Photos 3 and 4).

8.3 - Development of an approach for the study of density of hydraulic fills

According to the discussion above, a segregating flow on a hydraulic fill beach will create bedforms. The optimum flow conditions to maximize fill density seem to be the ones that form upper-stage plane beds. Possibly the lower limit of antidune flows, before significant flow separation occurs, may be also acceptable. Based on this, a rational design method to maximize fill density would include two main stages:

1 - Determination of the flow conditions that create the optimum bedform for the material that will be deposited and

2 - Determination of the appropriate discharge parameters that will produce on the beach the flow conditions determined from the previous stage.

The next sections of this chapter discuss the two design stages described above. Section 8.3.1 describes some of the criteria for occurrence of bedforms as a function of the flow conditions, that are found in the fluvial hydraulics literature and in the sedimentology literature. It also discusses how these methods could lead to the development of a similar criterion for hydraulic fills. Section 8.3.2 comments upon the potential use of this type of criterion on the design of hydraulic fills. Section 8.3.3 draws some ideas on a possible approach for the second stage, the design of the discharge method in such a way that the appropriate flow conditions occur on the beach.

8.3.1 - Criteria for occurrence of bedforms

There are several criteria for occurrence of bedforms as a function of the flow conditions presented in the literature. These criteria differ basically in the assumptions that are made and in the variables that are analyzed. The final product of each method is usually a diagram where the domain of the various bed forms is defined. This diagram is called by sedimentologists "bed form phase diagram" and can be used to determine in which bed form domain a particular flow will be placed.

These criteria were developed mainly for rivers and channels that usually have much deeper flows with a much smaller sediment load than flows on hydraulic fill beaches. Due to lack of field data, many bed form phase diagrams were based on flume data, but still with values of flow depths and sediment concentration very different from hydraulic fills. Another point to be considered is that these criteria were developed for equilibrium or quasi-equilibrium situations, while hydraulic fills feature intense aggradation. Therefore, these diagrams are not expected to be directly applicable to hydraulic deposition. However, the concept of using such type of diagrams for analysis of hydraulic fills is still valid. If developed for the appropriate conditions of flow and sediment transport, a criterion of this kind could be used to define the ideal flow parameters for the formation of an adequate fill.

Southard (1971) proposed a particularly useful type of bed form phase diagram, which besides being simple and making use of parameters that are already familiar to geotechnical engineers, has other advantages that will be discussed later.Southard (1971) adopts the following set of variables to define the interaction between fluid flow and sediment transport:

d - mean depth of flow
V - mean velocity of flow
ρ - density of fluid
μ - viscosity of fluid
D_{50} - mean size of sediment
ρ_s - density of sediment
g - acceleration due to gravity

The adoption of this set of variables implies that sorting (or coefficient of uniformity C_U) and grain shape are not of primary importance, although it is known that they are not negligible (Harms et al., 1982).

Using the principles of dimensional analysis, these seven variables can be expressed as a set of four dimensionless parameters. Several different sets of dimensionless parameters are possible and would describe the configuration of the bed equally well. A typical set of parameters in hydraulic engineering would consist of a Reynolds number, a Froude number, a size ratio and a density ratio. However, Southard (1971) proposes a different set that seems to be more convenient in the particular case of hydraulic fills:

$$\frac{\rho_s}{\rho} \qquad V\left[\frac{\rho}{\mu\, g}\right]^{\frac{1}{3}} \qquad d\left[\frac{\rho^2 g}{\mu^2}\right]^{\frac{1}{3}} \qquad D_{50}\left[\frac{\rho^2 g}{\mu^2}\right]^{\frac{1}{3}}$$

The convenience of this set of dimensionless parameters is that the critical variables (density, velocity, depth and particle size) are separated and each one appears in only one of the parameters. Thus these parameters can be viewed as dimensionless expressions of density (ρ_s), velocity (V), depth (d) and particle size (D_{50}), respectively. Also when comparing data for quartz sand and water over a limited range of temperatures, the terms in parenthesis will be constant. Therefore, the bed configuration can be expressed directly as a function of mean flow velocity (V), mean flow depth (d) and mean grain size (D_{50}), which makes the analysis of the flow conditions directly from the diagram easier.

Adopting these parameters, the bed form phase diagram for quartz sand and water (over a limited range of temperature) is the three dimensional plot of mean flow velocity (V) versus mean flow depth (d) and versus mean grain size (D_{50}), with delimitation of the zones of occurrence of each bed form. An example of this three dimensional plot is shown in Figure 8.5a, where the grain size (D_{50}) in on the vertical axis, flow depth (d) is on the horizontal axis on the plane of the page and flow velocity (V) is on the horizontal axis oblique to the page. Cross-sections can be taken at convenient positions. Due to the spatial variation in shape of the domain of the various bed forms, different projections will show different shapes for the various domains. Examples of three possible cross-sections are illustrated in Figure 8.5a: a horizontal section of flow velocity (V) versus flow depth (d) for D_{50}= 0.5 mm, and two vertical sections of flow velocity (V) versus grain size (D_{50}) for flow depths 0.2 and 20 m. The most convenient cross section for hydraulic fills is mean flow velocity (*V*) versus mean flow depth (*d*) for a particular grain size (D_{50}), which is usually known. An example of such diagram is shown in Figure 8.5b. Assuming that the flow on hydraulic fill beaches is qualitatively similar to the flow in the flume tests presented in the literature, this velocity versus depth plot would have the format indicated in Figure 8.6. This type of diagram could be used for the design of hydraulic fills, as describe in the next item.

8.3.2 - Use of the concept of a bed form phase diagram for hydraulic fills

Although further experimental study is still necessary to determine the geotechnical properties of sedimentary deposits formed under different flow conditions, it is already possible to anticipate that the flow conditions that will deposit the optimum hydraulic fill might be in the domain of flows that generate upper-stage plane beds. Assuming that a bed form phase diagram of the format presented on Figure 8.6 can be developed for hydraulic fills, this section will discuss how this diagram could be used for analysis of hydraulic fills.

The domains of "no movement" and "small ripples" never happen under typical hydraulic fill situations, therefore, once adapted for hydraulic fills analysis, this diagram would have its origin closer to the boundary between dunes and upper-stage plane beds (Figure 8.6). The area of possible optimum fill density is shaded in the diagram. Accordingly, hydraulic fills should be designed in such a way that the flow over the beach falls in this area.

In this diagram, lines of constant specific flow rate (total flow rate divided by flow width) plot as parallel straight lines as shown in Figure 8.6. Lines of constant Froude number also plot as straight lines and follow the same general orientation as the boundary between the upper-stage plane bed and the antidune domains. The domain of upper-stage plane beds opens up for higher specific flow rates, so the higher the specific flow rate, the higher the chance of being in the range of depths and velocity where a high density fill will be obtained. This observation is consistent with the results obtained experimentally Chapters 3 and 5, in which higher specific flow rates produced higher densities. It is also consistent with the Soviet experience that calls for maximization of the specific flow rate in order to obtain higher densities (Yufin, 1965; Melent'ev et al., 1973). The Soviet technology also recommends that the slurry concentration used be relatively low to improve density, which has also been verified experimentally in Chapters 3 and 5. In the diagram of Figure 8.6, the slurry concentration (and sediment transport rate) along the lines of equal specific flow rate increases to the right, therefore the probable domain of higher densities occurs at the lower end of the values of slurry concentration for hydraulic fills, i.e., the shaded area. In addition, the Soviet minimize velocity and upper-stage plane beds cover exactly the lower range of velocities within the domain of hydraulic fill conditions. Therefore, at least qualitatively this type of diagram is able to provide a rational explanation for the empirical evidence.

Concluding, a bed form phase diagram of this type developed for hydraulic fills could be used to determine the values of flow depth and flow velocity for each grain size that would produce a fill with optimum geotechnical parameters.

8.3.3 - Determination of the discharge parameters

Once a bed form phase diagram is developed for hydraulic deposition of a particular grain size, the flow conditions on the beach that will produce the optimum fill are known. Subsequently, it would be necessary to impose these conditions by creating a flow on the beach such that its depth and velocity correspond to the domain of upper-stage plane beds.

The next stage of the design would then consist of the determination of the discharge parameters such as flow rate, slurry concentration and discharge velocity, that produce the ideal flow conditions on the beach.

8.4 - Comments

Despite the importance of the density of hydraulic fills for the performance of structures, there is no rational method to design the density of these fills. This chapter indicates how the problem of density of hydraulic fills could be analyzed from the point of view of the interaction between fluid flow and sediment transport.

Since the question of density of hydraulic fills consists basically of studying a geotechnical property of materials deposited by flows, it becomes necessary to define the interface between the geotechnical problem and the hydrodynamic problem.

Flows over erodible surfaces create bed forms on the surface, which in turn alter the flow and the sediment transport and deposition mechanisms. Different deposition mechanisms generate distinct sedimentary deposits. Bed forms are then the result of the flow sediment transport interaction and also provide indications of the existing type of sedimentary structure. Therefore, the study of bed forms constitutes the link between the hydrodynamic and the geotechnical aspects of the hydraulic deposition phenomenon.

Knowledge of this link makes the problem solvable. From the geotechnical point of view, it becomes necessary to determine the properties of the different deposits associated with the various bed forms. There are several indications that upper-stage plane beds constitute the most favourable situation, but further research in this area is needed. From the hydrodynamic side, it seems to be necessary an adaptation of the bedform phase diagram for the range of parameters typical of hydraulic fills

The approach presented here points at flow depth and flow velocity on the beach as relevant parameters of hydraulic deposition. It also concludes that the initial density can be optimized by improving the discharge method, a concept that has been used for years in the Soviet Union with positive results.

This chapter suggests an approach the question of density of hydraulic fills, but further research is necessary to develop these ideas.

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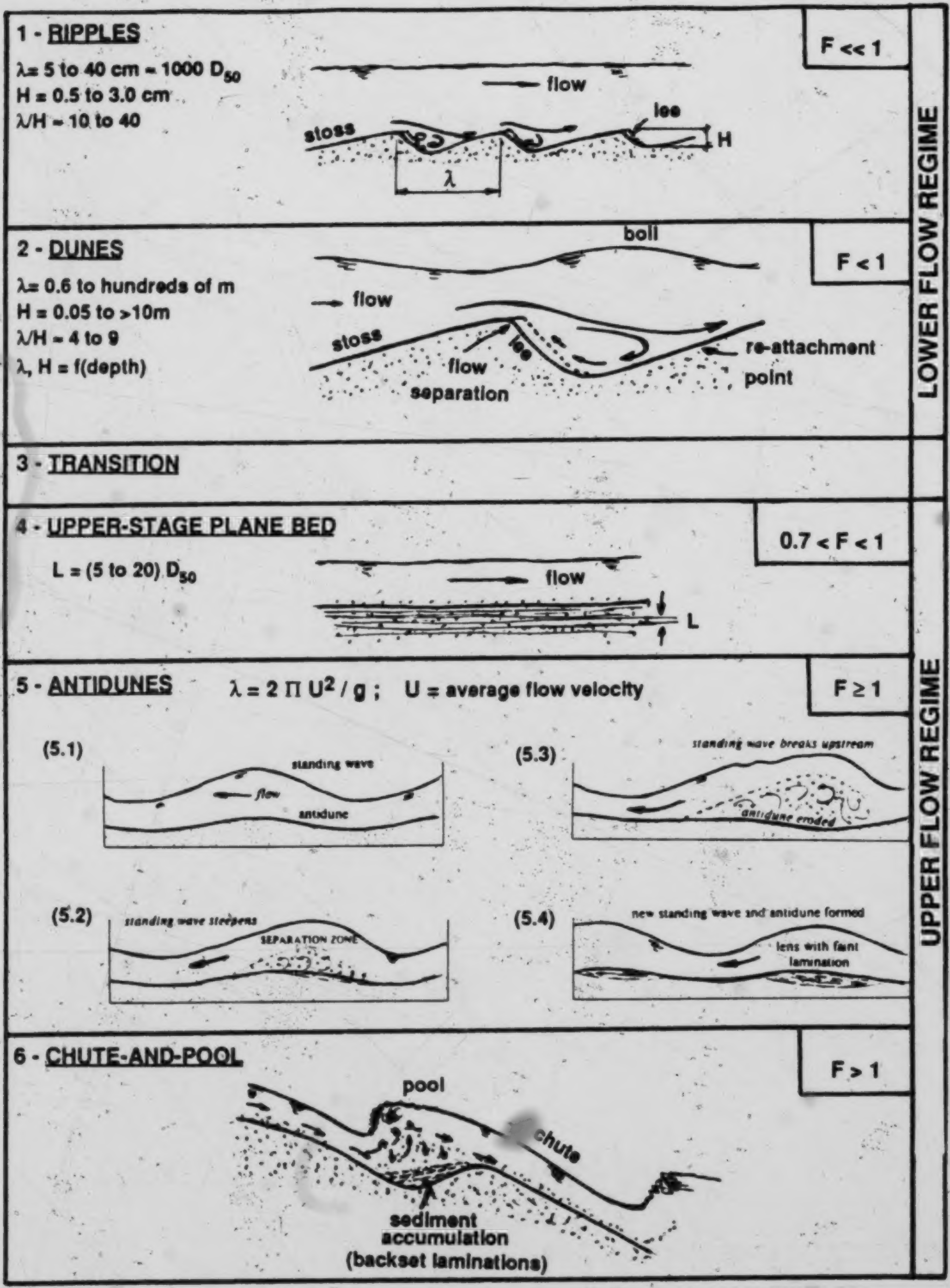
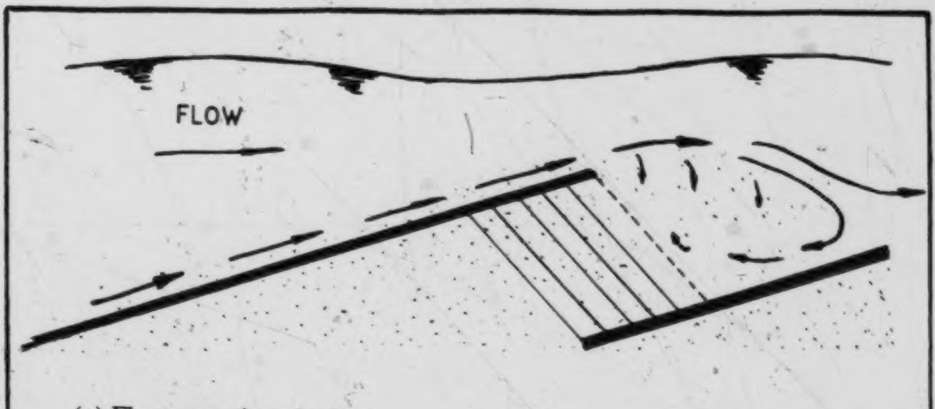
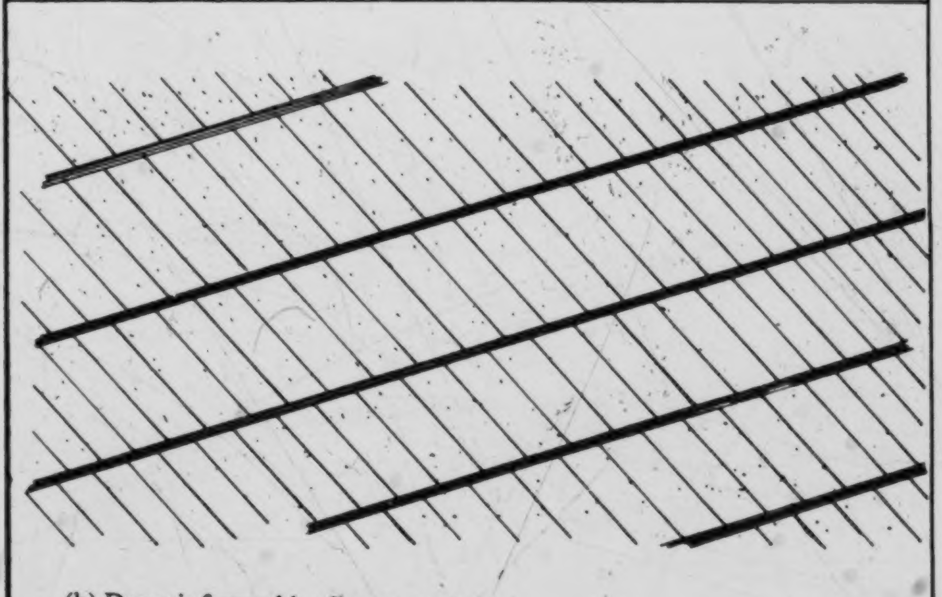

1 - RIPPLES
λ= 5 to 40 cm ≈ 1000 D50
H = 0.5 to 3.0 cm
λ/H ≈ 10 to 40
flow
lee
stoss
H
λ
F << 1
2 - DUNES
λ= 0.6 to hundreds of m
H = 0.05 to >10m
λ/H ≈ 4 to 9
λ, H = f(depth)
boil
flow
stoss
lee
flow
separation
re-attachment
point
F < 1
LOWER FLOW REGIME
3 - TRANSITION
4 - UPPER-STAGE PLANE BED
L = (5 to 20) D50
flow
L
0.7 < F < 1
5 - ANTIDUNES
λ = 2 Π U² / g ; U = average flow velocity
F ≥ 1
(5.1)
standing wave
flow
antidune
(5.2)
standing wave steepens
SEPARATION ZONE
(5.3)
standing wave breaks upstream
antidune eroded
(5.4)
new standing wave and antidune formed
lens with faint lamination
6 - CHUTE-AND-POOL
pool
chute
sediment
accumulation
(backset laminations)
F > 1
UPPER FLOW REGIME

FIG. 8.1 - Sequence of bedforms for increasing flow strength



(a) Flow over dune bed



(b) Deposit formed by flow over dune

FIRM
VERY SOFT

FIG. 8.2 - Flow and resulting stratigraphy under dune conditions

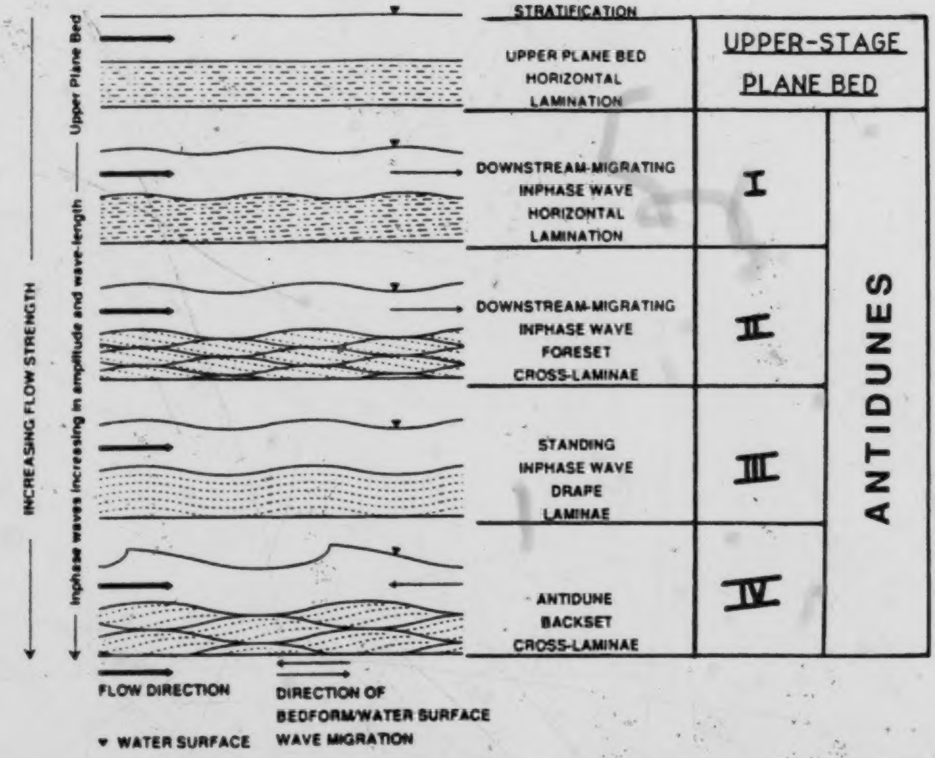

STRATIFICATION
UPPER-STAGE PLANE BED
UPPER PLANE BED HORIZONTAL LAMINATION
DOWNSTREAM-MIGRATING INPHASE WAVE HORIZONTAL LAMINATION
I
DOWNSTREAM-MIGRATING INPHASE WAVE FORESET CROSS-LAMINAE
II
STANDING INPHASE WAVE DRAPE LAMINAE
III
ANTIDUNE BACKSET CROSS-LAMINAE
IV
ANTIDUNES
INCREASING FLOW STRENGTH
Inphase waves increasing in amplitude and wave-length
Upper Plane Bed
FLOW DIRECTION
▾ WATER SURFACE
DIRECTION OF BEDFORM/WATER SURFACE WAVE MIGRATION

FIG. 8.3 - Schematic illustration of bed phases and stratification formed under upper-stage plane beds for the four energy levels of antidunes (modified after Cheel, 1990)

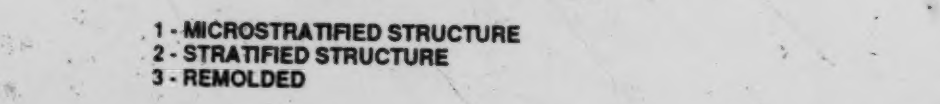
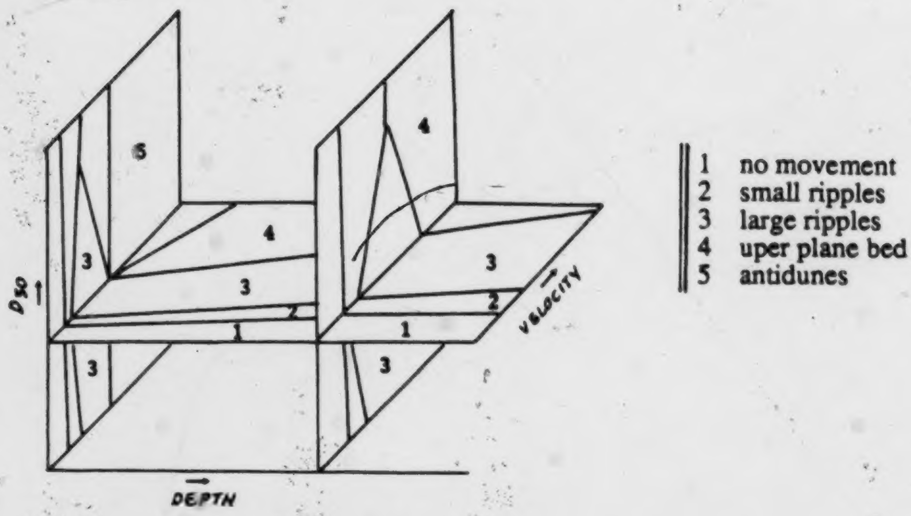
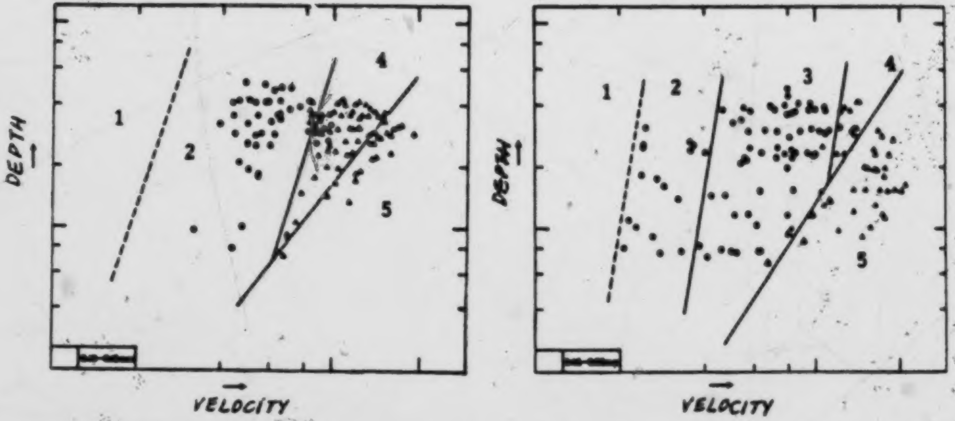


FIG. 8.4 - Variation of density with type of stratification and slurry concentration (modified after Yufin, 1965)


D50
DEPTH
VELOCITY
5
4
3
3
2
1
4
3
2
1
3
3
1 no movement
2 small ripples
3 large ripples
4 uper plane bed
5 antidunes

(a) Generalized depth-velocity-grain size diagram (modified after Rubin and McCulloch, 1980)



(b) Velocity vs. depth projections (modified after Harms et al., 1982)

FIG. 8.5 - Velocity vs. depth vs. grain size bedform phase diagrams

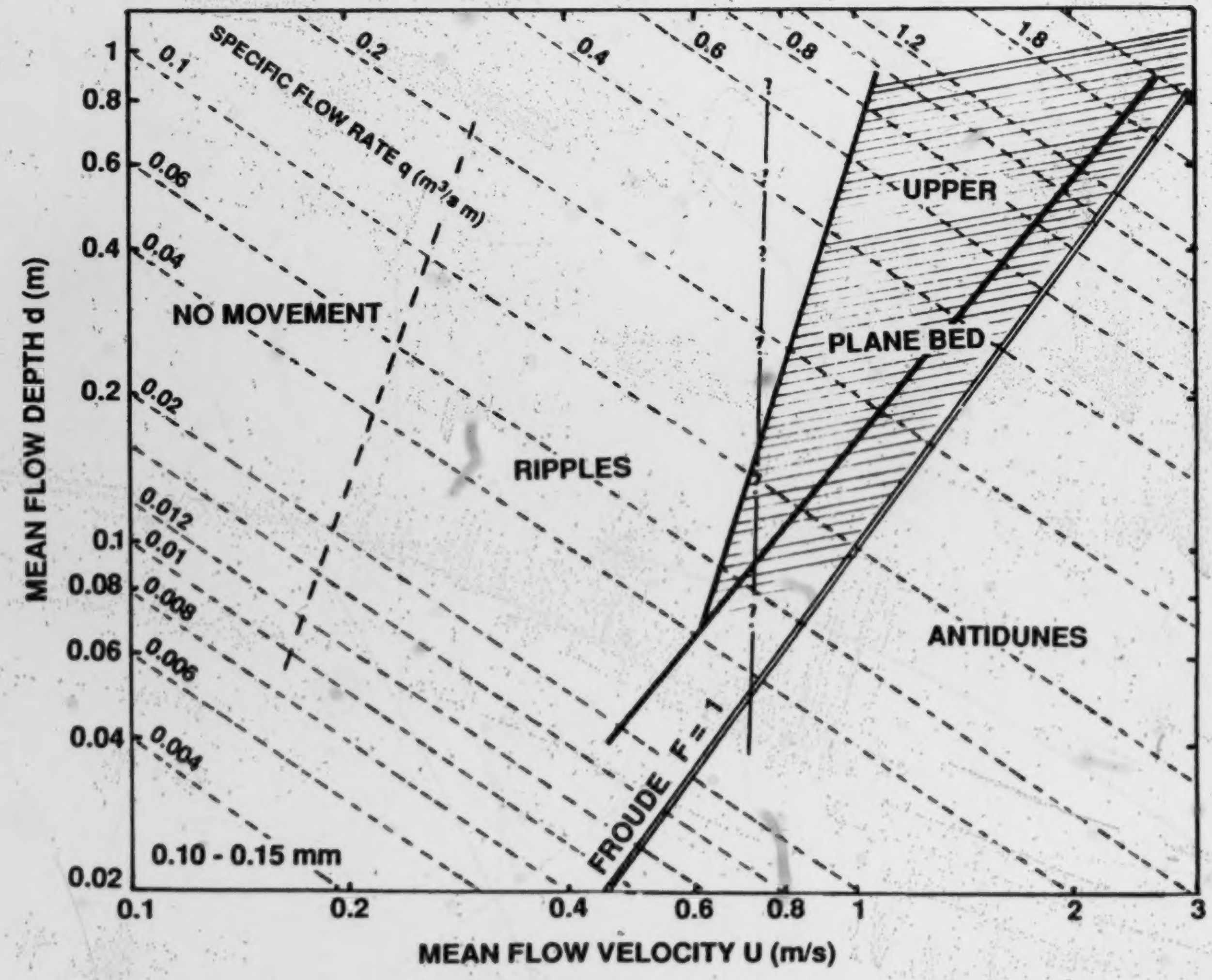


FIG. 8.6 - Bedform phase diagram for D_{50} = 0.10 to 0.15 mm (domains for low concentration)

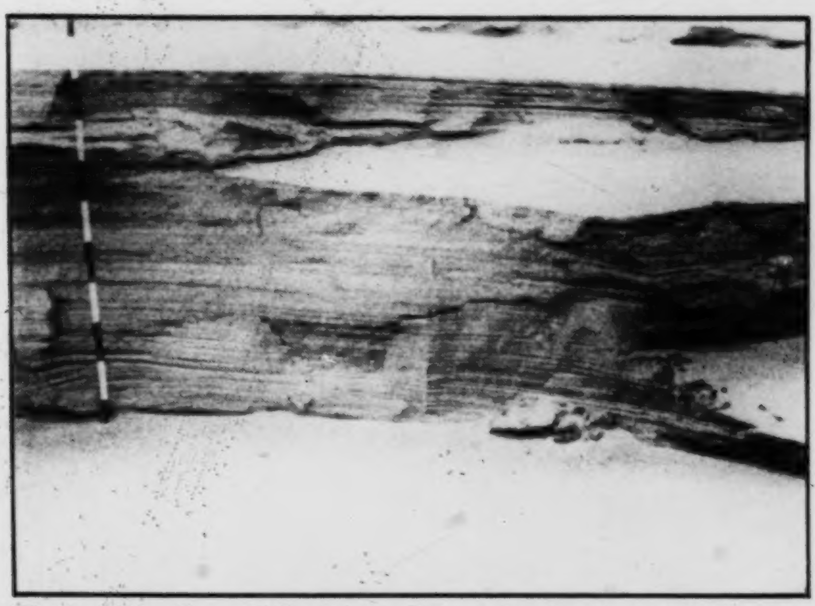


PHOTO 8.1 - Flat bedding (from "Alluvial Fans" by A.Rachocki, ©1981 John Wiley & Sons, Ltd. Reprinted by permission of John Wiley & Sons, Ltd.)

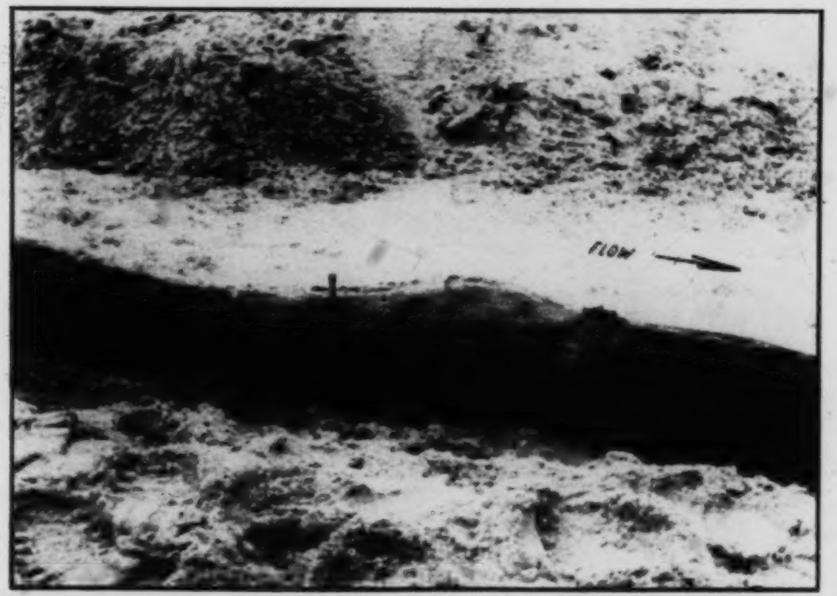


PHOTO 8.2 - Antidune bedding (from "Alluvial Fans" by A.Rachocki, ©1981 John Wiley & Sons, Ltd. Reprinted by permission of John Wiley & Sons, Ltd.)

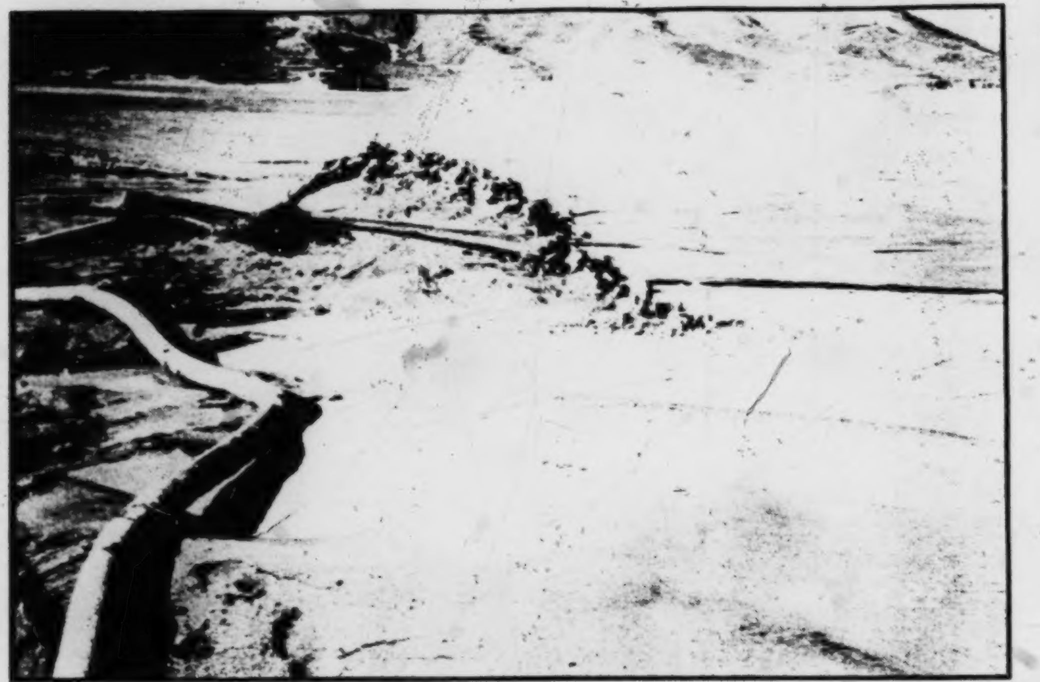


PHOTO 8.3 - Discharge point and flow conditions at Dashihe tailings dam (China)

PHOTO 8.4 - Sheet flow turning into braided flow; upper-stage plane bed with isolated trains of antidunes towards the downstream (Dashihe tailings dam, China)

Chapter 9

Conclusions

Hydraulic fill has many applications such as construction of water retention dams, tailings dams and artificial islands, disposal of waste material and backfilling of mines. The utilization of hydraulic fills in these cases is attractive due to practical and economical advantages of hydraulic fill over other methods. Among these advantages are the high rate of construction, high degree of mechanization, relatively low cost, applicability to a wide range of materials, promotion of particle separation and convenience in handling materials that are already in slurry form.

Despite its wide range of applicability, several aspects of hydraulic fill are not well understood, including the mechanism of fill formation and the factors that affect the properties of the fill. Consequently, the design of hydraulic fill tends to be limited to following previous experience, which does not always result in the safest and most economical fill. In addition, hydraulic filling may form soft or liquefiable deposits. Therefore, it is of interest to understand the deposition mechanisms in order to make it possible to promote adequate conditions to maximize density and enhance behaviour.

In this context, an experimental study of hydraulic fill was carried out to investigate the deposition process, the characteristics of the fill and the relationship between the mechanisms of fill formation and resulting properties.

The experimental study included laboratory flume deposition tests and field deposition tests. The flume tests were designed to model the field situation based on the similarity-of-process concept, due to the difficulties associated with the conventional hydraulic modelling of sediment transport phenomena. Input parameters such as slurry concentration and flow rate were varied and the characteristics of the fill were determined. Several tests were performed using three different sands; the results obtained were comparable to the results of other flume tests found in the literature. Large scale field tests were also carried out varying the composition of the slurry and monitoring the resulting fills. Qualitatively, the results of the field tests and the flume tests compared well, showing that the flume tests are a valuable tool to study the physical phenomena associated with hydraulic filling. Also, the flume tests proved to be powerful to isolate variables and help understanding the interaction among the variables in the hydraulic deposition process.

The main conclusions drawn from this experimental study and their relevance for the design of hydraulic fills are summarized below.

9.1 - DESIGN CONSIDERATIONS

Hydraulic fill is an engineering material that needs to be designed to perform adequately under the conditions required by each project. As for many other construction materials, the properties of hydraulic fill depend on the composition of the mix and on the placement method.

The composition of the mix in the case of hydraulic fills is defined by the slurry concentration, type of carrier fluid and type and grain size distribution of the solid fraction in the slurry. Whether the composition of the mix defines a slurry of segregating or a non-segregating behaviour seems to be the most critical factor. These two types of slurry behaviour generate distinct depositional conditions, with significant impact on the fill geometry, density and grain size distribution. Non-segregating slurries do not permit hydraulic sorting and produce a steeper beach of approximately constant granulometric characteristics and relatively low density. Segregating slurries deposit flatter and denser beaches with mean grain size varying with distance from the discharge point. These factors are significant for the performance of the hydraulic fill and must be considered in the design of the composition of the mix.

The placement method of hydraulic fills involves the depositional parameters such as slurry flow rate, discharge velocity, spacing, position and number of spigots and the details of the construction procedure. The depositional parameters determine the flow conditions, bedforms, drainage patterns and intensity of deposition, which affect the properties of the fill. Therefore, these parameters also need to be taken into consideration in the design of hydraulic fill.

Although each project has its own set of conditions, the important characteristics to be considered in most cases of hydraulic fill are the mechanical behaviour, which is related to the grain size distribution, fabric and density of the beach material, and the geometry. These factors and their relationship with the composition of the mix and the placement method were studied in some detail and the main findings are discussed below.

9.2 - GRAIN SIZE DISTRIBUTION AND FINES CAPTURE

The principal factor determining the grain size distribution along the beach is the slurry behaviour. A segregating slurry deposits the coarser fraction of the solids closer to the discharge point and the finer fraction farther away. Therefore, the mean grain size decreases and the coefficient of uniformity and the amount of fines increase with distance from the deposition point. This phenomenon of hydraulic sorting does not occur for non-segregating slurries, which form a hydraulic fill of approximately constant granulometric characteristics. Consequently, fines capture is maximum for non-segregating slurries and becomes less efficient as hydraulic sorting becomes more accentuated.

Hydraulic sorting is more pronounced for high flow rates, small slurry concentration and relatively low flow velocity on the beach (Yufin, 1965; Soderberg and Bush, 1977). Under these conditions segregation is enhanced and hindered effects are minimized, allowing the flow to interact with each grain individually. Moreover, with low flow velocity and small slurry concentration, sediment transport rate is relatively low, which also favours hydraulic sorting (Vanoni, 1975).

Therefore, the design of hydraulic fills for the cases in which sorting is important must consider the slurry behaviour and the sediment transport rate. The latter is of particular importance for the design of flume tests.

9.3 - DENSITY

Density is a key factor in the design of hydraulic fills, as it is essential for stability under static and dynamic conditions. Also, for waste disposal fills, an increase in density enhances the lifetime of the facility.

The study of density of hydraulic fills of relatively clean sands faces practical difficulties related to the accurate determination of the undisturbed density of the fill. This fact associated with the nature of the flow contribute to a relatively high scatter in the values of density obtained in the field and in the laboratory.

The results of flume tests show a decrease in density with increasing slurry concentration. This trend is consistent with the results reported by Yufin (1965) and Ferreira et al.(1980). Also, the densities obtained in the flume tests increased as the flow rate and the mean grain diameter increased. The trend in variation of density with slurry concentration for the field tests is not very clear, partially because other variables also affected the results.

The densities obtained in the experiments correspond to average relative densities of the order of 25 to 55%. As the maximum and minimum densities were shown to be sensitive to small variations in grain size distribution, the average relative densities were calculated taking this into consideration.

An approach to the control of density of hydraulic fills is suggested, based on the hydraulics of the flow and its interaction with the solid material being transported and deposited. This interaction between the flow and the sand leads to the formation of different bedforms depending on the flow conditions and on the sand grain size. Evidence from the hydraulic studies of bedforms, alluvial fan studies and field observation of hydraulic fill provides consistent indication that the densest fills might be formed under the same conditions that produce upper-stage plane beds. These concepts are discussed in the thesis, but the experimental verification is left as a recommendation for future research.

9.4- FABRIC

A fabric analysis of undisturbed samples taken from the flume and the field experimental fills was carried out using the Scanning Electron Microscope.

The presence of clay or clay conglomerate connectors between the sand grains was observed in the field samples. The material discharged in the flume had been processed and did not contain an appreciable amount of fines, therefore clay connectors were not developed.

The study showed that there is a tendency for grain alignment approximately parallel to the flow direction. The assessment of the effects of fabric on the behaviour of hydraulic fill is recommended as well as a more detailed study of the possible variations of fabric. Both flume and field samples displayed similar fabrics in relation to grain orientation. This observation reinforces the potential of flume tests in the study of hydraulic fills.

9.5- GEOMETRY

Beach geometry plays an important role in the design of hydraulic fills as it controls several factors such as fill volume, duration of construction, position and size of the pond, location of decant facilities, lay-out and area of the structure, storm water storage capacity and costs, among other aspects. In the case of water storage facilities, beach geometry also affects the required size of the starter dam, the rate of crest rise and the construction schedule.

Generally the beach profile is a smooth concave curve. A normalized profile has been shown to be properly described by a power equation (Melent'ev et al., 1973) or by an exponential equation (Smith, 1984; Smith et al., 1986). The application of these expressions for the design of hydraulic fill requires the knowledge of the beach length and the value of an exponent n . Blight et al.(1985) found that n was independent of scale and therefore could be determined from laboratory flume tests. However, flume and field tests carried out for this thesis did not confirm this. Wates et al.(1987) also did not find a constant value of n as postulated by Blight et al.(1985).

Fan (1989) and Fan and Masliyah (1990) develop a non-linear model for beach aggradation based on the basic flow equations and on a sediment transport equation. The model successfully described the geometry of flume deposits as a function of time. It is the most promising approach for the study of geometry of hydraulic fills, and further research is recommended to calibrate the model to the range of parameters and to the boundary conditions typical of hydraulic fills

The overall beach slope of hydraulic fills increases for larger slurry concentrations and for coarser materials and decreases as slurry flow rate increases. These trends were consistent in all cases of flume and field observations, including the ones from the literature. The same behaviour has also been observed for materials deposited hydraulically in nature.

An empirical parameter (P') is proposed to estimate the average slope. For the sets of data that were analyzed, this parameter P' seems to normalize flume and field slopes, which permits the transference of slopes determined in a flume to the field scale. It also seems to allow a preliminary estimate of the beach overall slope that could be used at the design stage, before field results are available. More research is necessary to determine the physical basis of this parameter (if any) and its limitations.

9.6- OTHER ASPECTS

A deep channelling phenomenon has been recognized and described. Deep channelling halts beach deposition and consequently construction as all the material is conveyed to the pond and this can have important consequences for the construction schedule and for the operation of decant facilities. The cause of deep channelling was discussed based on the studies of a similar phenomenon in alluvial fans and was verified by simulation in the laboratory flume. Deep channelling is caused by a lack of equilibrium between the beach slope and the input parameters, emphasizing the importance of maintaining the composition and flow rate of slurry approximately constant during the discharge process.

The stratification of the flow in the pipeline was shown to be an additional element that can be used in the planning and optimization of a hydraulic deposition scheme. The same is valid for the diameter, position and spacing of spigots.

A study of the interaction between the subaqueous and the subaerial environments could also lead to the development of pond management strategies to enhance beach deposition.

9.7- CONCLUDING REMARKS

The design of hydraulic fill requires the consideration of several aspects of the fill such as sorting, fines capture, density and geometry. However, it is not possible to optimize all these parameters simultaneously and a decision must be made in relation to which parameters are more relevant in each case, depending on the particular conditions of each project. It seems that the only parameter that can be changed without conflict is the flow velocity, as a minimization of the velocity appears to beneficial to most, if not all, aspects of hydraulic fill.

The importance of the flow conditions on the beach cannot be over-emphasized. The flow is responsible for transporting and depositing the grains and as such is the building agent, comparable to the type of equipment and number of passes for compacted fills. It is essential to have this information in order to assess the energy level under which the fill was formed and consequently, the fill properties. As more data on the flow characteristics becomes available, its effect on the resulting fill properties should become clearer.

9.8- REFERENCES

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APPENDIX A - LABORATORY EQUIPMENT AND TEST PROCEDURE

A.1 - LABORATORY EQUIPMENT

The laboratory equipment consists basically of a flume and its feeding and drainage systems. Figures A.1 and A.2 present sketches of the overall laboratory set-up, that will be described in more detail on the following paragraphs.

The flume (1) (Figures A.1, A.2) is 6.1 m (20 ft) long, 0.6 m (2 ft) wide and 1.2 m (4 ft) deep with Plexiglass walls and aluminum bottom, supported by a steel structure. The structural design of the flume is presented in Figures A.3 to A.6. The steel structure has two longitudinal support beams, vertical posts every 0.6 m and angle cross-beams at the top and at the bottom. All joints were sealed with silicone. The area between two vertical posts was called a panel and the panels were numbered from 1 to 9 starting from the upstream end. Each panel had a grid drawn on it. The flume is divided lengthwise in half by a removable Plexiglass wall (2). This dividing wall allows to run a test in one side of the flume while the material deposited on the other side is draining or being sampled or being prepared for the next run. It also allows for variation of the width of the flume being used, by moving the dividing wall sideways or by removing it.

The slurry is formed by feeding dry sand to a water stream. The water comes from a constant head reservoir (3) equipped with a float (4) and an overflow (5) for extra safety. From the reservoir (3), the water goes through a flowmeter (6) into a chute (7) that feeds the flume (1). The flowmeter is a variable area glass tube flowmeter for in-line installation and instantaneous readings (model # 1114-10H4A1A, Brooks Instrument Division, Emerson Electric Co.). A valve on the flowmeter allows to control the flow within the range of 1 to 40 l/min. The flowmeter was carefully calibrated after being installed.

Dry sand is stored outside the building in large bags (8) of 2 to 4 tons each and provisions were made to prevent the sand from getting wet. Each sand bag has an opening at the bottom, which is placed over a stainless steel funnel connected to a stainless steel pipe that feeds the hopper of a sand feeder (9). This feeder (Vibra Screw SCR-20) consists of a hopper that feeds sand to a rotating auger. Both the hopper and the auger are associated with vibration systems. The feed rate is adjustable on a dial and the range of rates can be altered by changing the trough that encases the auger, or the auger, or by adjusting the amplitude of vibration. The feeder was calibrated for each of the sands used in the tests and for every combination of trough/auger/vibration utilized. Measurements of feeding rates performed during the tests have shown that once set, the rate remains fairly constant, even after several hours of continuous operation. The range of feeding rates used for the tests was 1.3 to 44.5 g/s.

The sand is fed at a constant rate in the chute (7) onto the water stream. Sand and water mix by turbulence on the way down to the discharge point forming the slurry. Independent water and sand feeding systems have the advantage of being simple and providing slurry of constant composition for an indefinite period of time. Any adjustments on the flow rate of sand and/or water can easily be done prior to and during a test. This method avoids the drawbacks associated with mixing tanks. The chute (7) feeds slurry to a flow spreader (10). This device spreads the flow uniformly across the width of the flume. The design of the flow spreader is presented on Figures A.7 to A.10. Thin metal vanes inside the flow spreader assure an even distribution of the flow and direct the flow lines parallel to the flume walls. The spreader was designed to create an one-dimensional discharge and to minimize the effects of the walls by having the flow parallel to a hydraulically smooth wall. Observation of the operation of the spreader indicates it was successful in this matter.

The flow spreader (10) is connected to the chute (7) by a flexible hose (11) and a funnel (12). The spreader is hung up by aircraft cable (13) on the shaft of a variable speed electric motor (14) placed above the flume. A floater (15) and a switch (16) automatically turn the motor on and off in order to keep the distance between the flow spreader (10) and the rising sand fill (17) constant and equal to a pre-determined value during the whole test. The feeding system just described can be used in either side of the flume.

At the downstream end of the flume, drainage is provided by a constant level drain (18) with adjustable height. A system of boards and screens (19) causes the head loss necessary to settle most of the solids still in suspension and help simulating the hydraulic fill's pond. Drainage from the flume can be shut down by closing the valve on the pipe that connects the drain (18) to the laboratory sump (20).

A.2 - TEST PROCEDURE

The first step of a test is to make the surface of the sand deposited on a previous test uniform. The initial sand surface can be of any slope but it should be smooth to avoid flow diversions. If the initial surface is too flat for the imposed flow conditions, preferred deposition at the upstream side will steepen the profile. However, if the initial slope is too steep, the flow will erode on the upstream end and deposit on the downstream side until the slope is flattened to its equilibrium value.

The second step is to position the spreader, the floater, the switch and the chute. The water flow is turned on and its flow rate is adjusted and measured. The flow of water over the sand in the flume accommodates the sand surface just prepared, settling down soft spots and eroding high areas. It also helps re-saturating the sand fill (foundation) and accumulating water upstream of the spreader for the floating switch (Figure A.2). The sand surface is then traced on transparent acetate sheets taped on each of the panels. This is the base line for the test that will be carried out. After this, the sand feeder dial is adjusted to the required rate and the feeder is turned on.

The slurry begins to be formed and the fill starts to rise. Periodically, the sand surface and the time were recorded on the transparent sheets. A test is considered completed when the new sand deposit has a minimum thickness of 20 cm on the first 6 panels and the surface slope is in equilibrium (at least 3 consecutive tracings are approximately parallel). The slurry is then sampled for verification of flow rate and composition (concentration and grain size distribution of the solid fraction), and both flows of water and sand are shut down. The test final profile is traced on the transparent sheets.

The deposited sand is allowed to drain for some hours before being sampled. For each test at least one undisturbed sample per panel and one remolded sample per post were taken. Undisturbed samples were used for density determination, triaxial tests and fabric study, while remolded samples were used for grain size distribution analysis.

Undisturbed samples were taken by statically pushing 3" or 4" sampling tubes into the sand bed. In order to study the disturbance caused by this sampling method, half a sampling tube was pushed into the sand bed contiguously to the flume transparent wall. By analyzing the bending of the thin layers of sand, it was conservatively assumed that the outer 1 cm of the sample in contact with the sampler was disturbed. After the sampling tube was pushed in, the sample was slowly frozen from bottom up using dry ice (carbon dioxide pellets) placed around the bottom of the sampler. A few samples that presented heave after being frozen were discarded. The use of metal samplers induced more frost heave. Due to the high thermal conductivity of the metal, the sample freezes faster and not only from the bottom up but also from the outside in. Better results were obtained with thin-walled plastic samplers. After completely frozen, the samples were removed from the sampler and in a walk-in freezer (T = -25° C), the outer ring of supposedly disturbed material was trimmed off. The samples were labelled and kept frozen until needed.

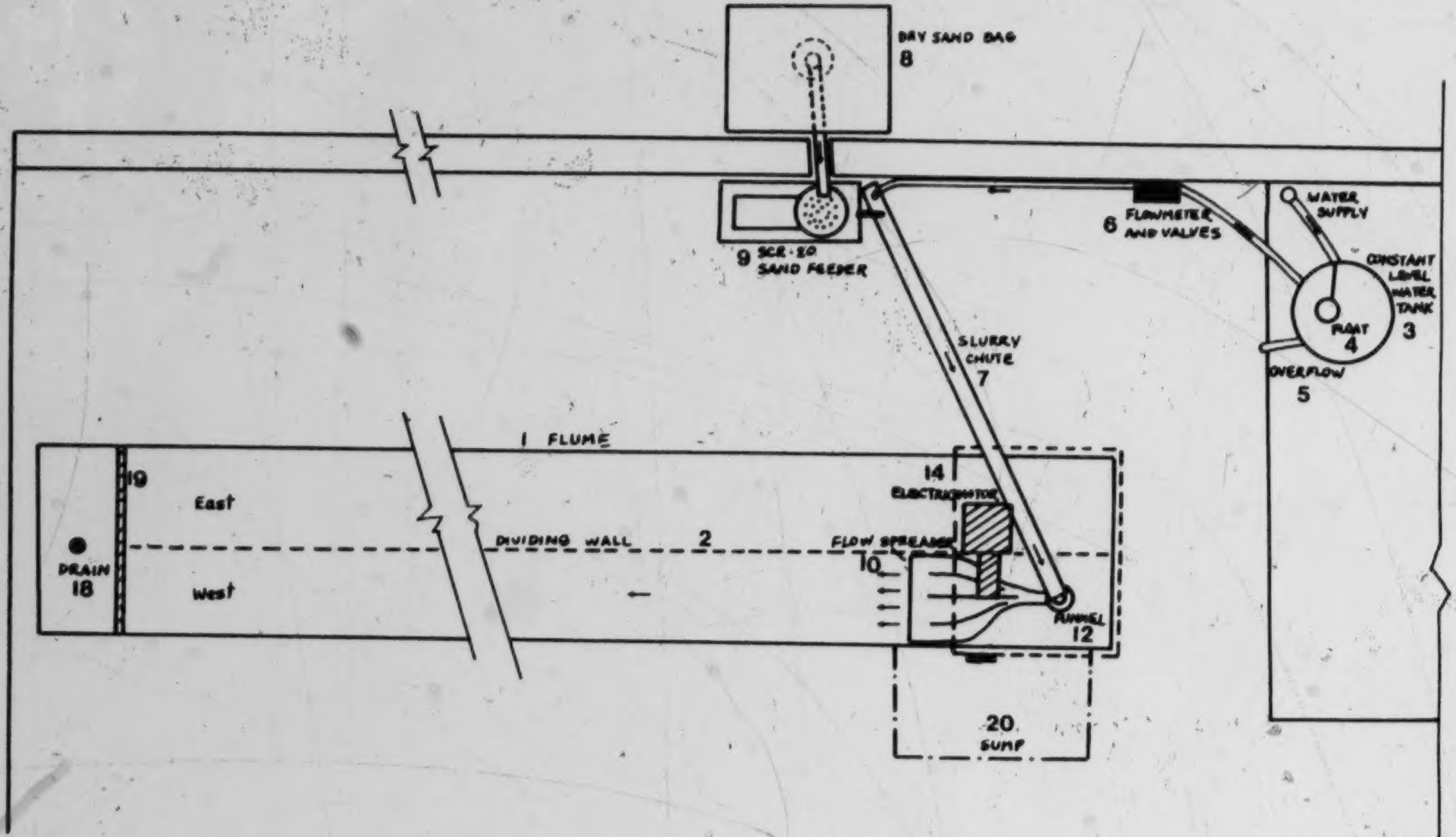

DRY SAND BAG
8
9 SCR-20 SAND FEEDER
6 FLOWMETER AND VALVES
WATER SUPPLY
CONSTANT LEVEL WATER TANK 3
FLOAT 4
OVERFLOW 5
SLURRY CHUTE 7
1 FLUME
14 ELECTROMOTOR
FLOW SPREADER
10
FUNNEL 12
20 SUMP
19
East
West
DIVIDING WALL 2
DRAIN 18

FIG. A.1 - Sketch of the flume set-up - Plan view (not to scale)

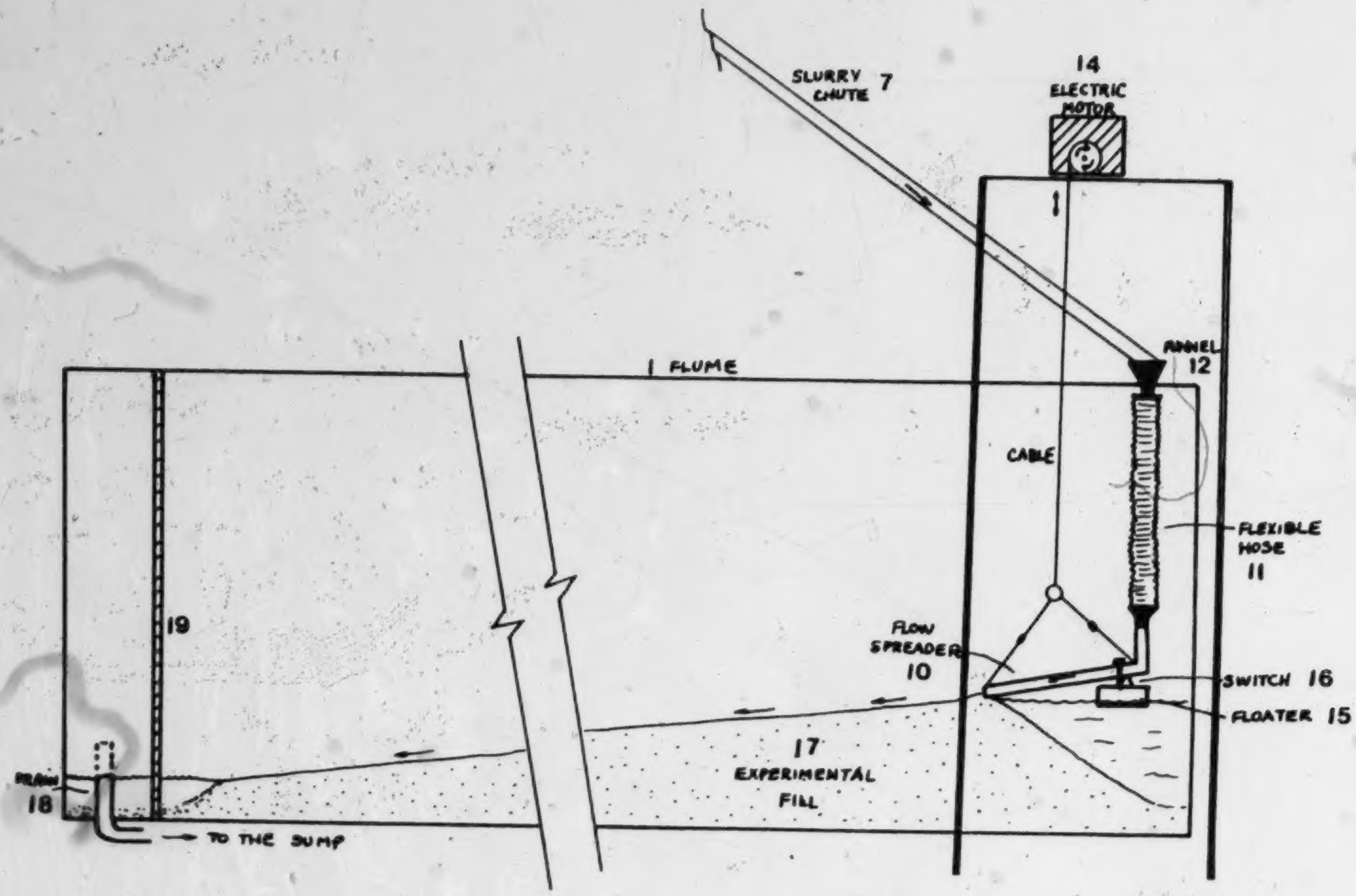

SLURRY CHUTE 7
14 ELECTRIC MOTOR
FUNNEL 12
1 FLUME
CABLE
FLEXIBLE HOSE 11
FLOW SPREADER 10
SWITCH 16
FLOATER 15
19
17 EXPERIMENTAL FILL
DRAIN 18
TO THE SUMP

FIG. A.2 - Sketch of the flume set-up - Lateral view (not to scale)

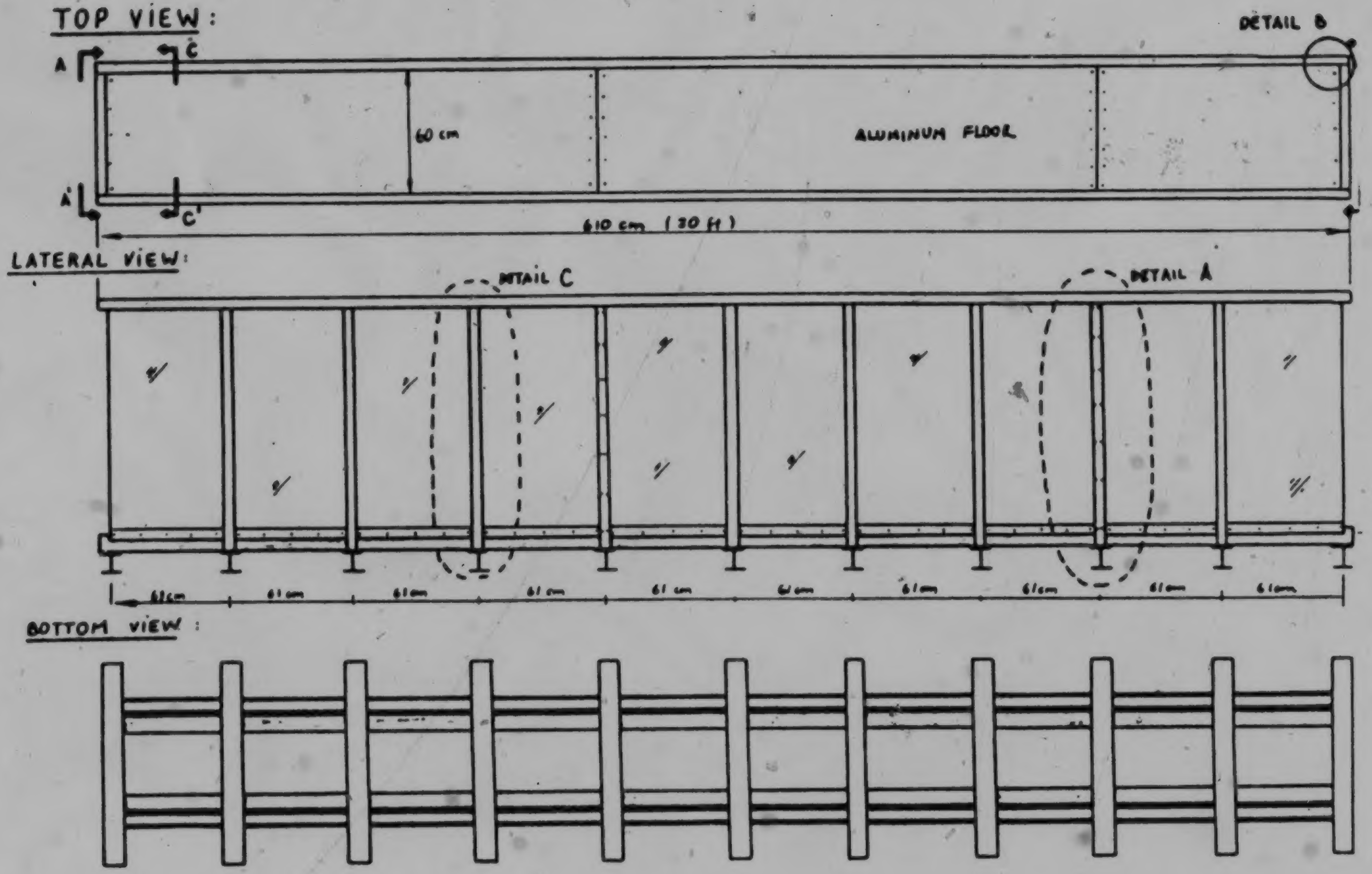

TOP VIEW:
A
C
A'
C'
60 cm
ALUMINUM FLOOR
DETAIL B
610 cm (20 ft)
LATERAL VIEW:
DETAIL C
DETAIL A
61 cm
61 cm
61 cm
61 cm
61 cm
61 cm
61 cm
61 cm
61 cm
61 cm
BOTTOM VIEW:

FIG. A.3 - Flume design

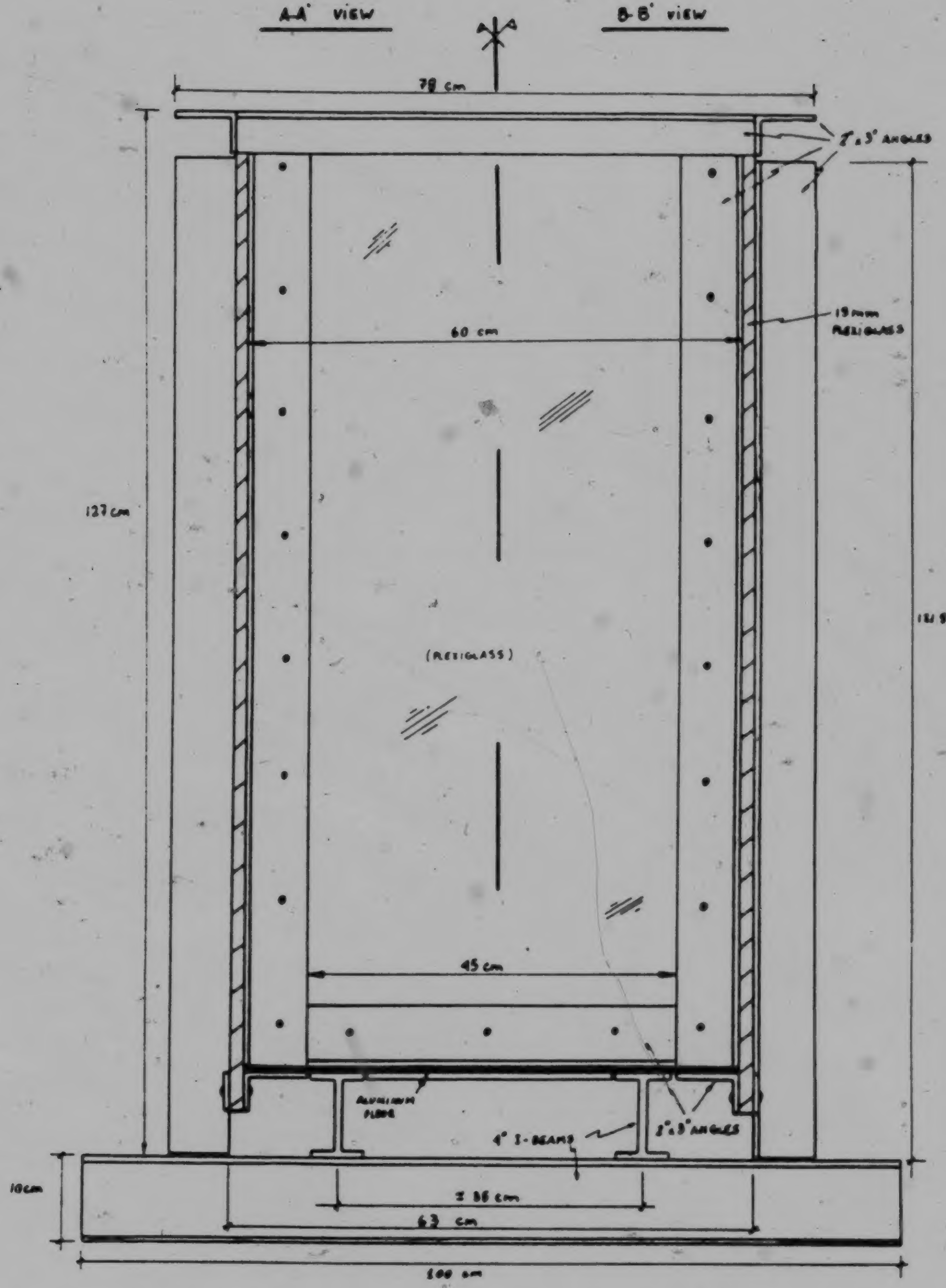

A-A' VIEW
B-B' VIEW
A A
78 cm
2" x 3" ANGLES
19 mm PLEXIGLASS
60 cm
127 cm
121.5 cm
(PLEXIGLASS)
45 cm
ALUMINIUM PLATE
4" I-BEAMS
2" x 3" ANGLES
10 cm
≈ 36 cm
63 cm
100 cm

FIG. A.4 - Views A-A' and B-B' (see Fig. A.3)

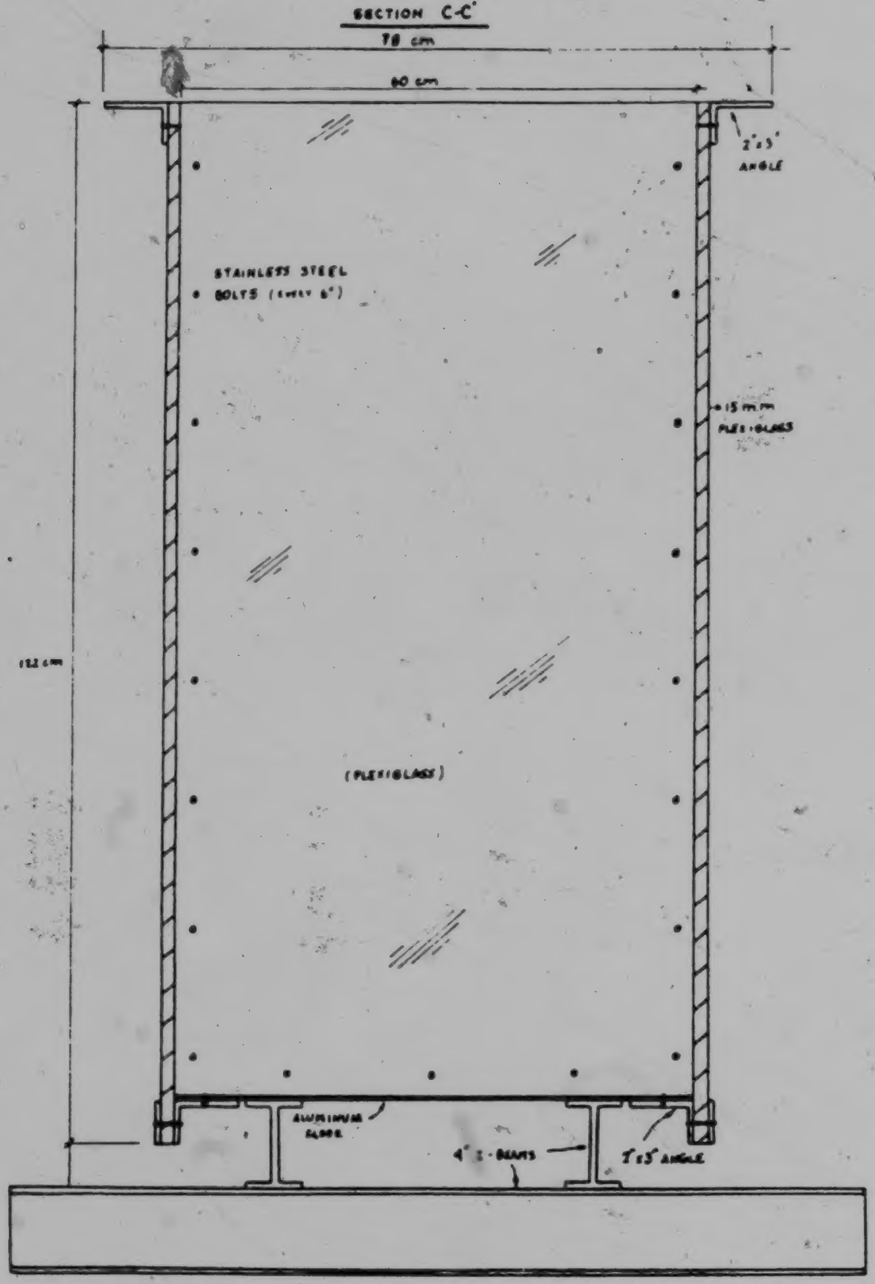

SECTION C-C'
78 cm
60 cm
2"x5" ANGLE
STAINLESS STEEL
BOLTS (EVERY 6")
15 mm
PLEXIGLASS
122 cm
(PLEXIGLASS)
ALUMINUM
FLOOR
4" I-BEAMS
2"x5" ANGLE

FIG. A.5 - Section C-C' (see Fig. A.3)

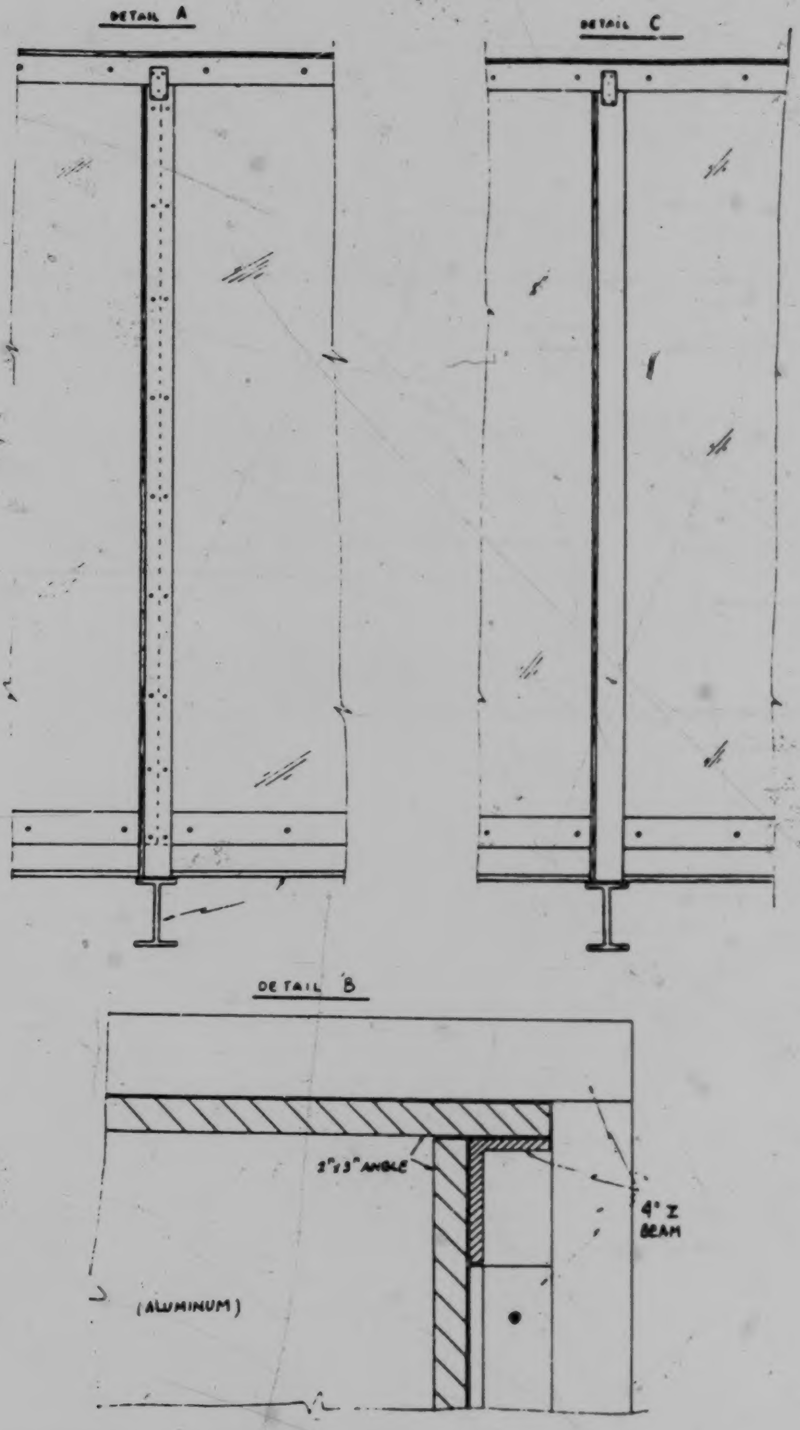

DETAIL A
DETAIL C
DETAIL B
2"x3" ANGLE
4" I BEAM
(ALUMINUM)

FIG. A.6- Flume details (see Fig. A.3)

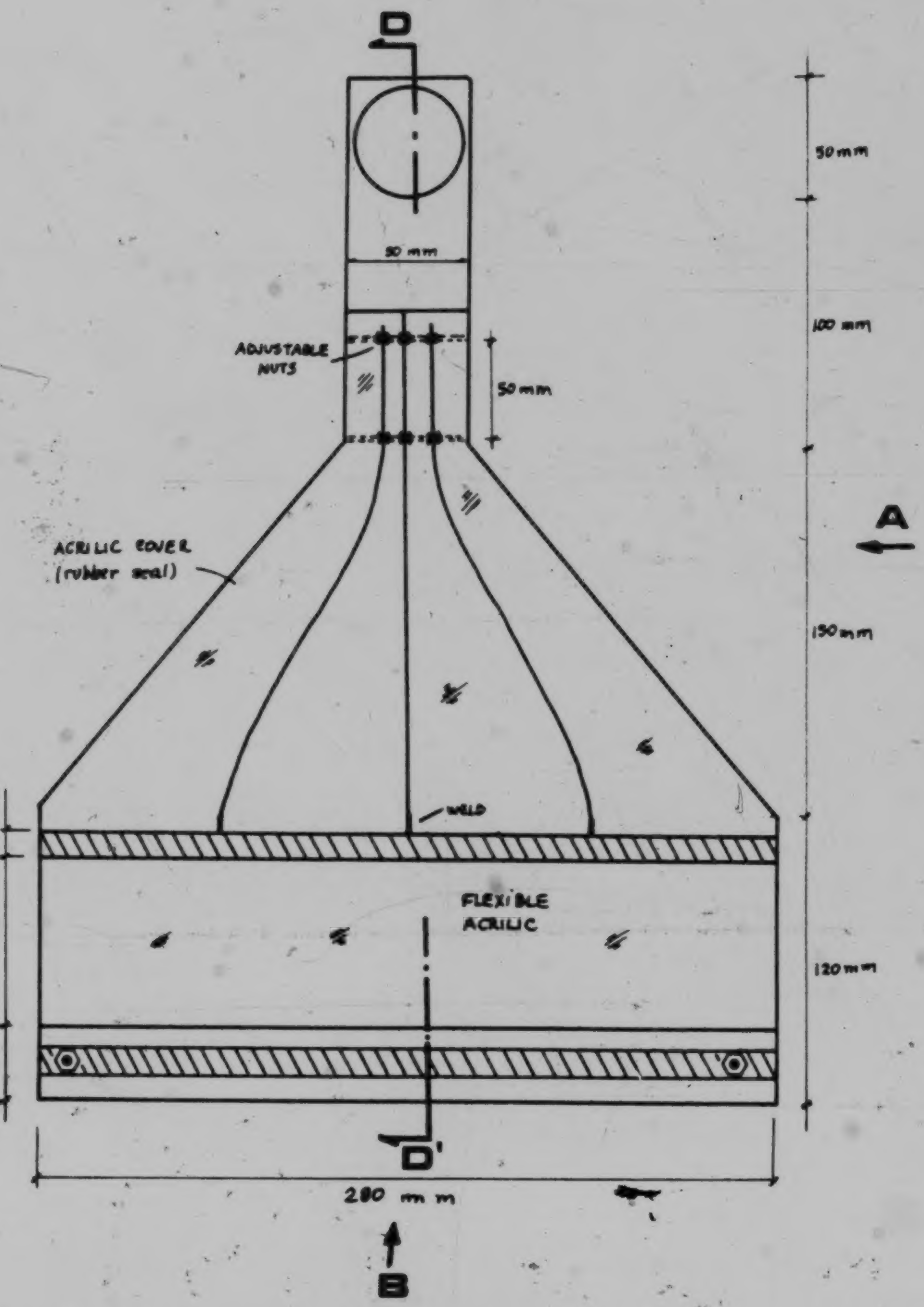

D
50mm
50 mm
100 mm
ADJUSTABLE NUTS
50mm
A
ACRILIC COVER (rubber seal)
150mm
WELD
10mm
FLEXIBLE ACRILIC
70mm
120mm
30mm
D'
280 mm
B

FIG. A.7 - Plan view of the flow spreader (not to scale)

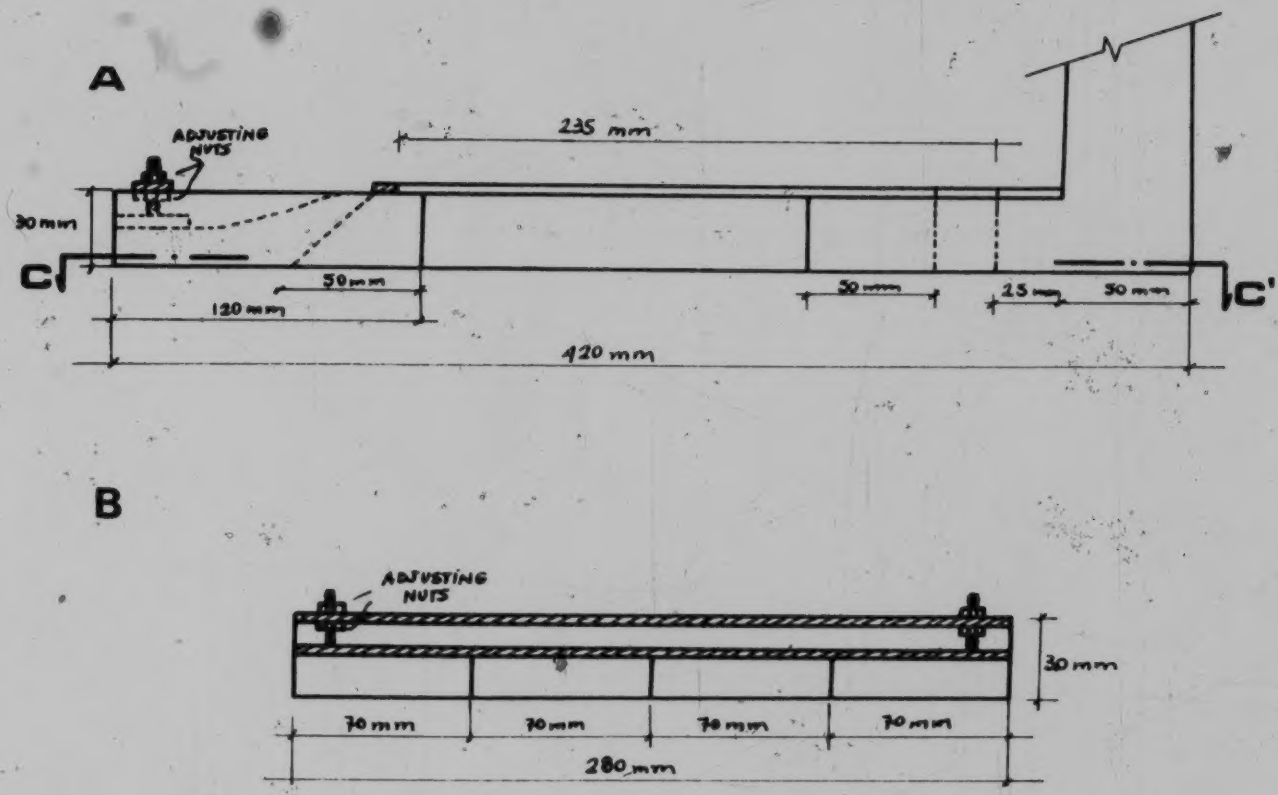

A
ADJUSTING NUTS
235 mm
30 mm
C
C'
50 mm
120 mm
50 mm
25 mm
50 mm
420 mm
B
ADJUSTING NUTS
30 mm
70 mm
70 mm
70 mm
70 mm
280 mm

FIG. A.8 - Lateral and frontal view of the flow spreader (not to scale)

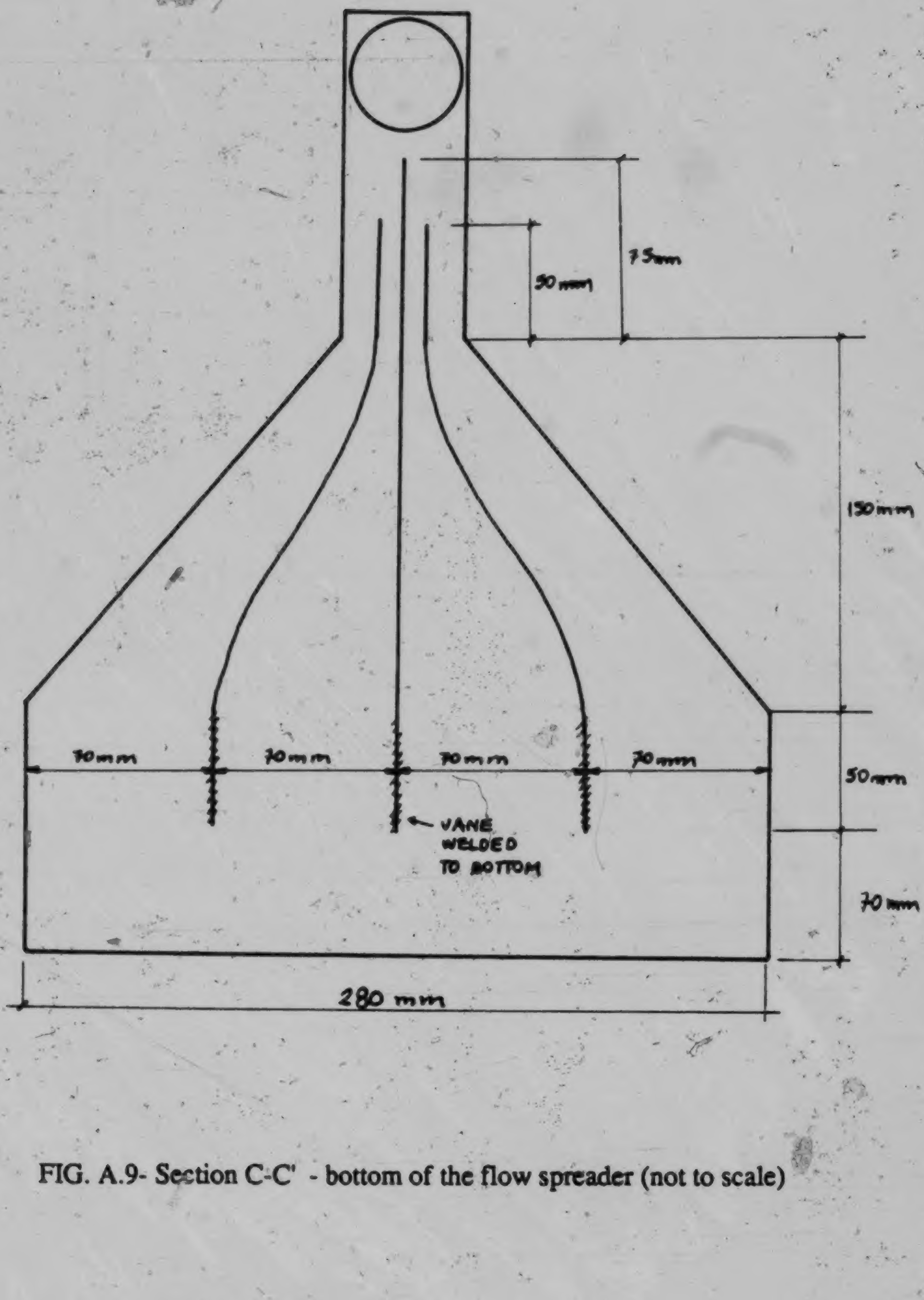

75mm
50 mm
150mm
70mm
70mm
70mm
70mm
50mm
VANE WELDED TO BOTTOM
70 mm
280 mm

FIG. A.9- Section C-C' - bottom of the flow spreader (not to scale)

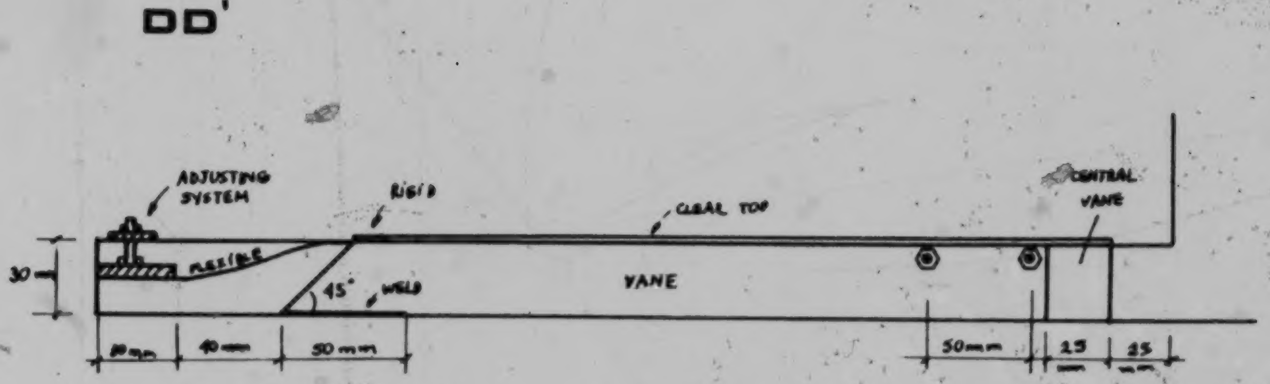

DD'
ADJUSTING SYSTEM
RIGID
CLEAR TOP
CENTRAL VANE
FLEXIBLE
45°
WELD
VANE
30
10 mm
40 mm
50 mm
50 mm
25 mm
25 mm

FIG. A.10 - Longitudinal cross section of the flow spreader (not to scale)

APPENDIX B - FLUME TESTS RESULTS

This appendix summarizes the results of the flume tests carried out to study hydraulic deposition. Three different sands were used: SS, KS and TS as described in Chapter 3. Tables B.1 and B.2 present a listing of all tests that were carried out and tables B.3 and B.4 give the values of slurry concentration, total flow rate, solids flow rate and water flow rate measured for each test.

The basic results of each test are plotted in this appendix and include:

- grain size distribution analysis of remolded samples taken at each panel (approximately every 60 cm along the flume). Due to the uniformity of the material extra sieves were added to the standard set to yield more accurate grain distribution curves. The set of sieves used included sieves number 10, 16, 20, 30, 40, 50, 60, 70, 80, 100, 120, 140, 170 and 200.

- profile of the deposit at different times during the tests, including the final profile.

- moisture content of undisturbed samples for various locations along the flume

- density of undisturbed samples for various locations along the flume, measured using a modified wax method on trimmed frozen specimens from the central part of the sample.

Table B.1 - SUMMARY OF SS AND KS FLUME TESTS

TEST No.	Cw nom (%)	Qt(v)nom (l/min)	H fall (cm)	Exit gap (mm)	Jet angle	Flume side	Start Date	time	Finish Date	time	Duration (hs.)	Comments
SS1	5.7	14.9	0	2.34	10	east	Aug 12,87	10:00	Aug 12,87	14:00	4.0	
SS2	3.5	14.8	0	2.34	10	west	Aug 18,87	11:00	Aug 20,87	14:10	13.7	
SS3	6	14.9	25	2.34	10	east	Aug 25,87	9:00	Aug 27,87	9:00	48.0	not continuous
SS4	3.5	14.8	25	2.34	10	west	-	-	-	-	-	not completed
SS5	6	14.9	25	4.57	10	east	Sept 23,87	8:30	Sept 23,87	14:10	5.7	
KS1	2	5	0	open	10	east	Aug 08,88	10:30	Aug 09,88	14:35	4.1	
KS2	6	5	0	open	10	east	Aug 10,88	9:40	Aug 11,88	8:00	22.3	not completed
KS3	6	5	0	open	10	east	Aug 11,88	10:00	Aug 11,88	22:10	12.2	
KS4	10	5	0	open	10	west	Aug 16,88	16:45	Aug 16,88	21:00	4.3	
KS5	15	5	0	open	10	west	Aug 18,88	9:00	Aug 18,88	13:40	4.7	
KS6	20	5	0	open	10	west	Aug 19,88	9:00	Aug 19,88	12:15	3.3	
KS7	25	5	0	open	10	east	Aug 19,88	14:00	Aug 19,88	17:10	3.2	
KS8	30	5	0	open	10	east	Aug 22,88	11:00	Aug 22,88	13:30	2.5	sand tongues
KS9	2	10	0	open	10	east	Aug 23,88	9:00	Aug 24,88	17:00	32.0	
KS10	6	10	0	open	10	east	Aug 25,88	14:00	Aug 25,88	21:30	7.5	
KS11	10	10	0	open	10	east	Aug 29,88	9:00	Aug 25,88	12:30	3.5	
KS12	15	10	0	open	10	east	Aug 29,88	13:15	Aug 29,88	15:10	1.9	
KS13	20	10	0	open	10	east	Aug 30,88	8:50	Aug 30,88	10:3	1.7	
KS14	10	20	0	open	10	east	Aug 30,88	11:30	Aug 30,88	13:35	2.1	
KS15	6	20	0	open	10	east	Aug 31,88	10:00	Aug 31,88	12:00	2.0	not completed
KS16	6	20	0	open	10	east	Sept 1,88	10:20	Sept 1,88	13:10	2.8	
KS17	2	20	0	open	10	east	Sept 2,88	13:20	Sept 2,88	23:10	9.8	
KS18	10	5	0	open	10	west	Dec 19,88	13:10	Dec 19,88	18:25	3.3	
KS19	10	5	0	open	10	west	Dec 20,88	13:30	Dec 20,88	16:20	2.8	water level drop

Table B.2 - SUMMARY OF TS FLUME TESTS

TEST No.	Cw nom (%)	Qt(v)nom (l/min)	H fall (cm)	Exit gap (mm)	Jet angle	Flume side	Start Date	time	Finish Date	time	Duration (hs.)	Comments
TS1	5.7	14.9	25	3.05	10	west	Dec 10,87	15:00	Dec 11,87	18:40	27.7	
TS2	10.7	12	0	open	10	west	Feb 24,88	10:00	Feb 24,88	13:00	3.0	
TS3	19.3	12	0	open	10	west	Mar 05,88	10:20	Mar 05,88	11:40	1.3	
TS4	19.3	12	34	open	10	west	Mar 09,88	10:30	Mar 09,88	12:00	1.5	
TS5	19.3	12	34	open	10	west	Mar 11,[illegible]	20:45	Mar 13,88	11:15	38.5	in 4 stages
TS6	2	5	0	open	10	east	Jun 02,88	15:00	Jun 03,88	12:00	21.0	
TS7	2	5	0	open	10	east	Jun 06,88	14:00	Jun 07,88	11:35	21.6	
TS8	6	5	0	open	10	east	Jun 07,88	14:00	Jun 07,88	21:50	7.9	
TS9	10	5	0	open	10	east	Jun 08,88	14:00	Jun 08,88	17:15	3.3	
TS10	15	5	0	open	10	east	Jun 14,88	16:30	Jun 14,88	20:15	3.7	
TS11	20	5	0	open	10	east	Jun 15,88	13:00	Jun 15,88	16:15	3.3	
TS12	25	5	0	open	10	west	Jun 16,88	15:45	Jun 16,88	18:40	2.9	
TS13	30	5	0	open	10	west	Jun 20,88	9:00	Jun 20,88	-	-	slurry didn't flow
TS14	10	10	0	open	10	west	Jun 20,88	12:50	Jun 20,88	17:15	4.4	
TS15	2	10	0	open	10	west	Jun 22,88	15:15	Jun 23,88	15:50	12.6	
TS16	6	10	0	open	10	west	Jun 24,88	11:45	Jun 24,88	17:00	5.3	
TS17	10	10	0	open	10	west	Jun 25,88	8:50	Jun 25,88	12:10	3.3	
TS18	30	5	0	open	10	west	Jun 25,88	14:00	Jun 25,88	16:20	2.3	
TS19	35	5	0	open	10	east	Jun 26,88	15:00	Jun 26,88	18:00	3.0	
TS20	15	10	0	open	10	east	Jun 27,88	11:30	Jun 27,88	13:30	2.0	
TS21	20	10	0	open	10	east	Jun 28,88	9:15	Jun 28,88	11:30	2.3	
TS22	15	15	0	open	10	west	Jun 28,88	14:00	Jun 28,88	15:40	1.7	
TS23	2	20	0	open	10	west	Jul 02,88	23:00	Jul 03,88	10:00	11.0	
TS24	10	20	0	open	10	west	Jul 04,88	11:00	Jul 04,88	13:30	2.5	
TS25	6	20	0	open	10	west	Jul 05,88	16:20	Jul 05,88	18:15	1.9	
TS26	10	5	0	open	10	west	Jul 06,88	12:50	Jul 06,88	17:20	4.5	
TS27	2	20	0	open	10	west	Jul 07,88	9:15	Jul 07,88	16:40	7.4	
TS28	10	15	0	open	10	west	Jul 09,88	10:45	Jul 09,88	14:30	3.8	
TS29	6	15	0	open	10	west	Jul 11,88	9:45	Jul 11,88	14:55	5.2	
TS30	40	5	0	open	10	east	Jul 12,88	13:35	Jul 12,88	16:00	2.4	
TS31	25	5	0	open	10	east	Jul 13,88	12:20	Jul 13,88	15:20	3.0	
TS32	6	10	0	open	10	east	Jul 14,88	9:00	Jul 14,88	17:00	8.0	
TS33	2	5	0	open	10	east	Jul 18,88	11:15	Jul 19,88	15:50	28.6	
TS34	30	5	0	open	10	east	Jul 21,88	10:00	Jul 21,88	12:15	2.3	
TS35	35	5	0	open	10	east	Jul 26,88	8:50	Jul 26,88	10:00	1.2	
TS36	40	5	0	open	10	east	Jul 28,88	13:30	Jul 28,88	16:50	1.3	
TS37	2	5	0	open	10	east	Aug 02,88	11:30	Aug 03,88	15:450	28.2	

Table B.3 - SUMMARY OF HYDRAULIC PARAMETERS OF TESTS SS AND KS

TEST No.	Cw nom (%)	Cw init (%)	Cw fin (%)	Cw aver (%)	Qt(v)nom (l/min)	Qt(v)fin (l/min)	Qt(w)fin (g/min)	Qs(v)nom (cm3/min)	Qs(v)fin (cm3/min)	Qs(w)nom (g/min)	Qs(w)fin (g/min)	Qw(v)nom (l/min)	Qw(v)init (l/min)	Qw(v)fin (l/min)
SS1	5.7	-	-	-	14.9	-	15464	326.00	-	864.0	-	14.60	-	-
SS2	3.5	-	-	-	14.8	-	15134	201.50	-	534.0	-	14.60	-	-
SS3	6.0	-	-	-	14.9	-	15464	326.00	-	864.0	-	14.60	-	-
SS4	3.5	-	-	-	14.8	-	15134	201.50	-	534.0	-	14.60	-	-
SS5	6.0	-	-	-	14.9	-	15464	326.00	-	864.0	-	14.60	-	-
KS1	2.0	2.4	3.7	3.0	5	3.2	3324	38.21	46.8	101.4	124.3	4.96	5.07	3.20
KS2	6.0	-	-	-	5	-	-	117.60	-	311.4	-	4.88	4.82	-
KS3	6.0	5.9	6.9	6.4	5	4.3	4503	117.60	118.1	311.4	313.0	4.88	4.97	4.19
KS4	10.0	9.5	9.4	9.5	5	5.0	5363	201.21	188.3	533.4	499.1	4.80	4.74	4.83
KS5	15.0	13.7	15.2	14.5	5	4.5	5047	312.17	284.2	827.4	753.0	4.69	4.74	4.22
KS6	20.0	19.3	20.0	19.7	5	4.7	5422	431.03	403.9	1142.4	1070.4	4.57	4.48	4.28
KS7	25.0	23.9	23.9	23.9	5	4.7	5671	558.66	521.4	1480.8	1381.8	4.44	4.41	4.19
KS8	30.0	28.5	29.1	28.8	5	4.9	6134	696.06	657.1	1844.4	1741.2	4.30	4.37	4.29
KS9	2.0	-	2.1	2.1	10	8.2	8303	76.42	64.8	202.8	171.6	9.92	9.96	8.10
KS10	6.0	6.0	7.1	6.5	10	8.3	8713	235.20	233.0	623.4	617.4	9.76	9.71	8.09
KS11	10.0	9.5	10.7	10.1	10	8.9	9576	402.41	385.6	1066.2	1021.8	9.60	9.74	8.51
KS12	15.0	13.2	13.3	13.3	10	9.9	10905	624.34	540.2	1534.8	1431.6	9.38	9.41	9.37
KS13	20.0	18.9	21.9	20.4	10	8.4	9835	862.07	804.9	2284.8	2133.0	9.14	9.13	7.55
KS14	10.0	9.0	9.0	9.0	20	20.9	22273	804.83	748.1	2133.0	1982.4	19.20	20.05	20.14
KS15	6.0	-	-	-	20	-	-	470.40	-	1246.8	-	19.53	19.31	-
KS16	6.0	5.7	5.8	5.8	20	19.4	20247	470.40	439.2	1246.8	1164.0	19.53	19.14	19.00
KS17	2.0	1.8	1.9	1.9	20	18.9	19195	152.85	137.9	405.0	365.4	19.85	19.97	18.79
KS18	10.0	10.0	14.0	12.0	5	3.3	3623	201.21	202.9	533.4	537.7	4.80	4.83	3.09
KS19	10.0	-	13.3	13.3	5	4.2	4483	201.21	227.7	533.4	603.4	4.80	-	3.95

Table B.4 - SUMMARY OF HYDRAULIC PARAMETERS OF FLUME TESTS TS

TEST No.	Cw nom (%)	Cw ini (%)	Cw final (%)	Cw aver (%)	Qt(v)nom (l/min)	Qt(v)fin (l/min)	Qt(w)fin (g/min)	Qs(v)nom (cm3/min)	Qs(v)fin (cm3/min)	Qs(w)nom (g/min)	Qs(w)fin (g/min)	Qw(v)nom (l/min)	Qw(v)init (l/min)	Qw(v)final (l/min)
TS1	5.7	-	-	-	14.9	14.9	15464	326.0	320.0	864.0	848	14.60	-	-
TS2	10.7	-	-	-	12	12.0	12880	520.8	-	1380.0	-	11.50	-	-
TS3	19.3	20.8	-	20.0	12	12.0	13634	994.0	-	2634.0	-	11.00	-	-
TS4	19.3	19.6	-	-	12	12.0	13634	994.0	-	2634.0	-	11.00	-	-
TS5	19.3	-	-	-	12	12.0	13634	994.0	-	2634.0	-	11.00	-	-
TS6	2.0	-	-	-	5	5.0	5061	38.2	38.2	101.4	101.4	4.96	4.96	-
TS7	2.0	1.6	2.5	2.0	3	3.2	3199	38.2	30.6	101.4	81.2	4.96	4.92	3.12
TS8	6.0	7.0	7.2	7.1	5	4.8	5053	117.6	138.0	311.4	365.7	4.88	4.85	4.69
TS9	10.0	10.2	10.3	10.2	5	5.1	5485	201.2	212.8	533.4	564.0	4.80	4.90	4.92
TS10	15.0	16.4	17.9	17.1	5	4.6	5199	312.2	350.2	827.4	928.0	4.69	4.73	4.27
TS11	20.0	21.6	21.4	21.5	5	5.1	5879	431.0	459.2	1142.4	1217.0	4.57	4.55	4.60
TS12	25.0	26.1	26.4	26.2	5	5.0	6011	558.7	599.6	1480.8	1589.0	4.44	4.49	4.42
TS13	30.0	-	-	-	-	-	-	696.1	-	1844.4	-	4.30	4.27	-
TS14	10.0	11.0	10.9	11.0	10	10.3	11051	402.4	453.1	1066.2	1200.7	9.60	9.76	9.85
TS15	2.0	1.6	1.9	1.8	10	8.5	8620	76.4	60.5	202.8	160.2	9.92	10.10	8.47
TS16	6.0	4.8	4.9	4.9	10	9.7	10077	235.2	187.1	623.4	495.8	9.76	9.75	9.51
TS17	10.0	10.2	10.3	10.2	10	10.1	10715	402.4	415.9	1066.2	1102.1	9.60	9.54	9.68
TS18	30.0	30.1	30.8	30.4	5	5.0	6110	696.1	697.2	1844.4	1847.6	4.30	4.30	4.29
TS19	35.0	33.9	33.9	33.9	5	4.9	6264	844.4	801.3	2238.0	2123.5	4.16	4.10	4.10
TS20	15.0	14.4	14.5	14.5	10	9.9	10856	624.3	595.6	1534.8	1578.4	9.38	9.36	9.33
TS21	20.0	19.6	19.8	19.7	10	10.0	11307	862.1	842.9	2284.8	2233.8	9.14	9.17	9.14
TS22	15.0	14.2	14.1	14.1	15	14.8	16356	936.5	870.4	2482.2	2306.5	14.06	13.95	13.95
TS23	2.0	-	0.0	-	20	-	-	152.9	-	405.0	-	19.85	19.81	-
TS24	10.0	8.8	8.6	8.7	20	20.4	21548	804.8	697.8	2133.0	1849.1	19.20	19.28	19.66
TS25	6.0	5.2	5.3	5.3	20	19.8	20275	470.4	403.4	1246.8	1069.0	19.53	19.43	19.41
TS26	10.0	8.6	8.5	8.5	5	5.0	5204	201.2	167.7	533.4	444.3	4.80	4.74	4.86
TS27	2.0	1.5	1.4	1.5	20	20.8	20846	152.9	110.3	405.0	292.4	19.85	19.66	20.72
TS28	10.0	7.6	7.8	7.7	15	14.8	15368	603.6	450.2	1599.6	1193.1	14.40	14.43	14.36
TS29	6.0	4.4	4.5	4.5	15	14.9	15302	352.8	260.5	934.8	690.3	14.65	14.82	14.66
TS30	40.0	29.9	29.8	29.9	5	4.8	5833	1005.0	656.5	2663.4	1739.6	4.00	4.07	4.11
TS31	25.0	18.0	17.9	18.0	5	4.8	5361	558.7	362.4	1480.8	960.3	4.44	4.37	4.39
TS32	6.0	3.9	4.2	4.1	10	9.2	9384	235.2	149.1	623.4	395.2	9.76	9.74	9.00
TS33	2.0	2.5	3.8	3.1	3	3.2	3343	38.2	47.9	101.4	126.9	4.96	4.99	3.19
TS34	30.0	30.9	31.2	31.0	5	4.9	6137	696.1	721.7	1844.4	1912.4	4.30	4.27	4.21
TS35	35.0	35.4	35.4	35.4	5	5.1	6551	844.4	874.2	2238.0	2316.5	4.16	4.22	4.22
TS36	40.0	40.3	40.1	40.2	5	5.0	6656	1005.0	1008.1	2663.4	2671.4	4.00	3.96	3.95
TS37	2.0	2.0	3.1	2.5	3	3.2	3283	38.2	38.0	101.4	100.8	4.96	5.03	3.15

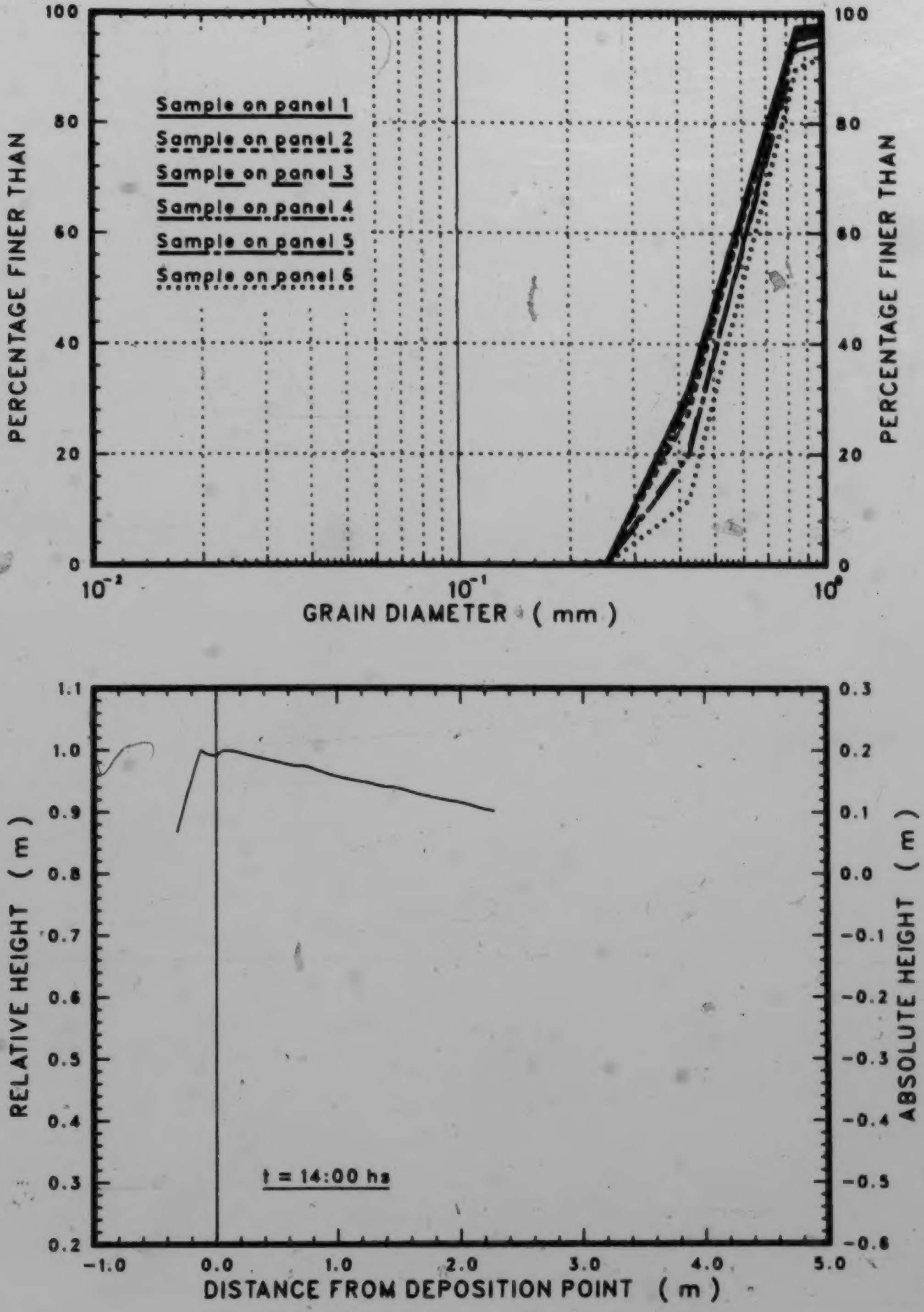


FIG. B.1 – Results of test SS1 (Qt = 15 l/min, Cw = 6 %)

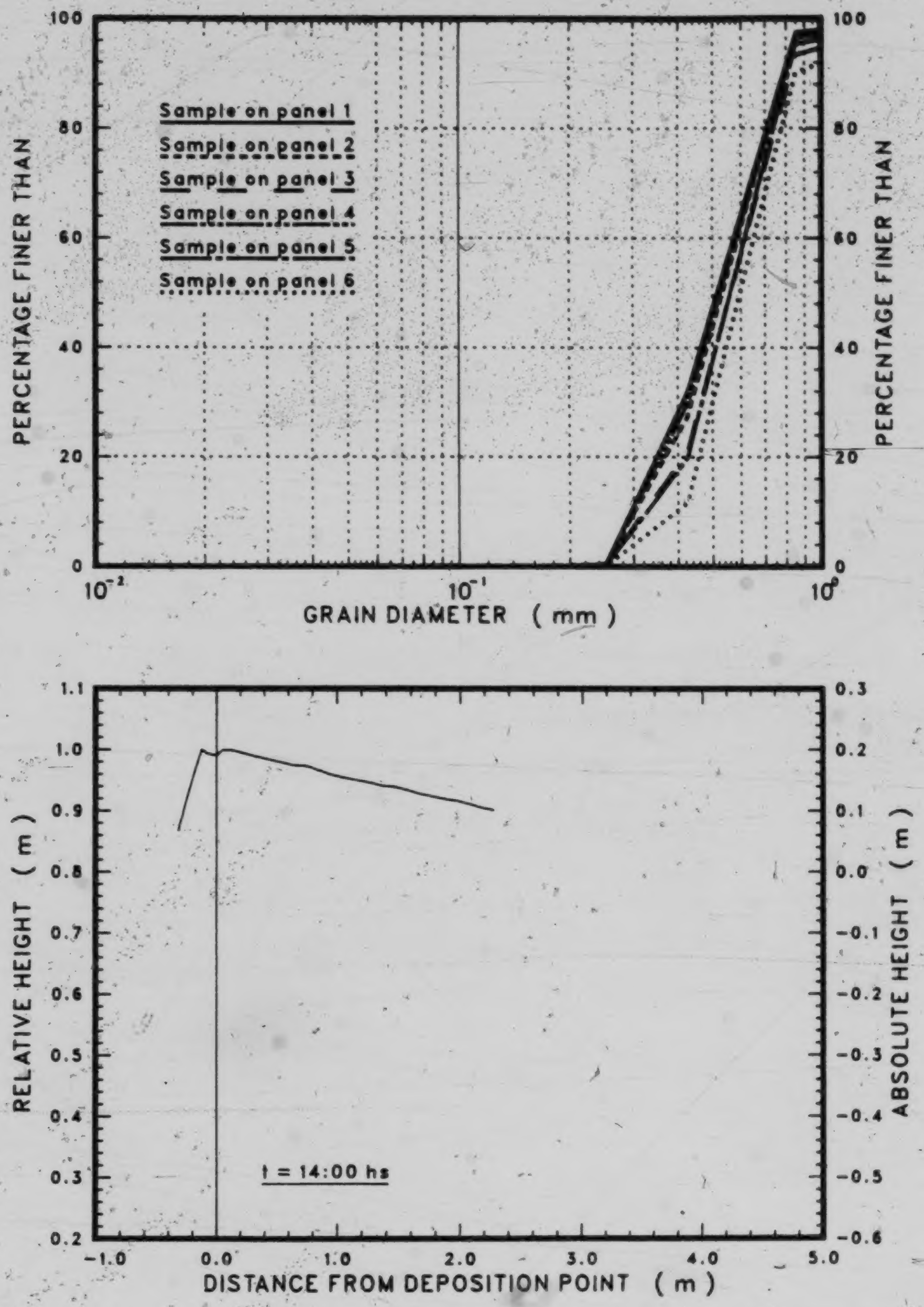


FIG. B.2 – Results of test SS1 (Qt = 15 l/min, Cw = 6 %)

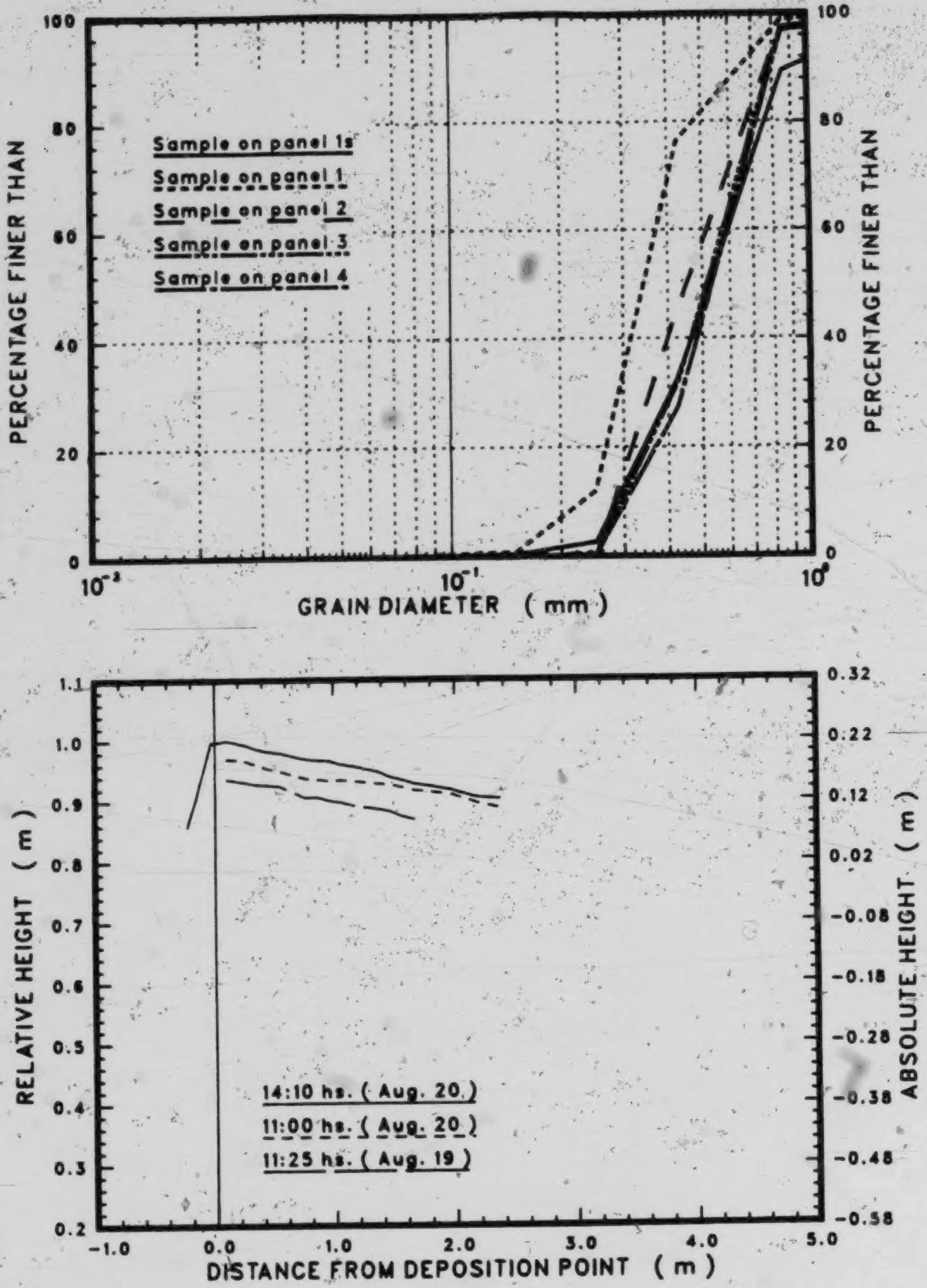


FIG. B.3 – Results of test SS2 (Qt = 15 l/min, Cw = 4 %)

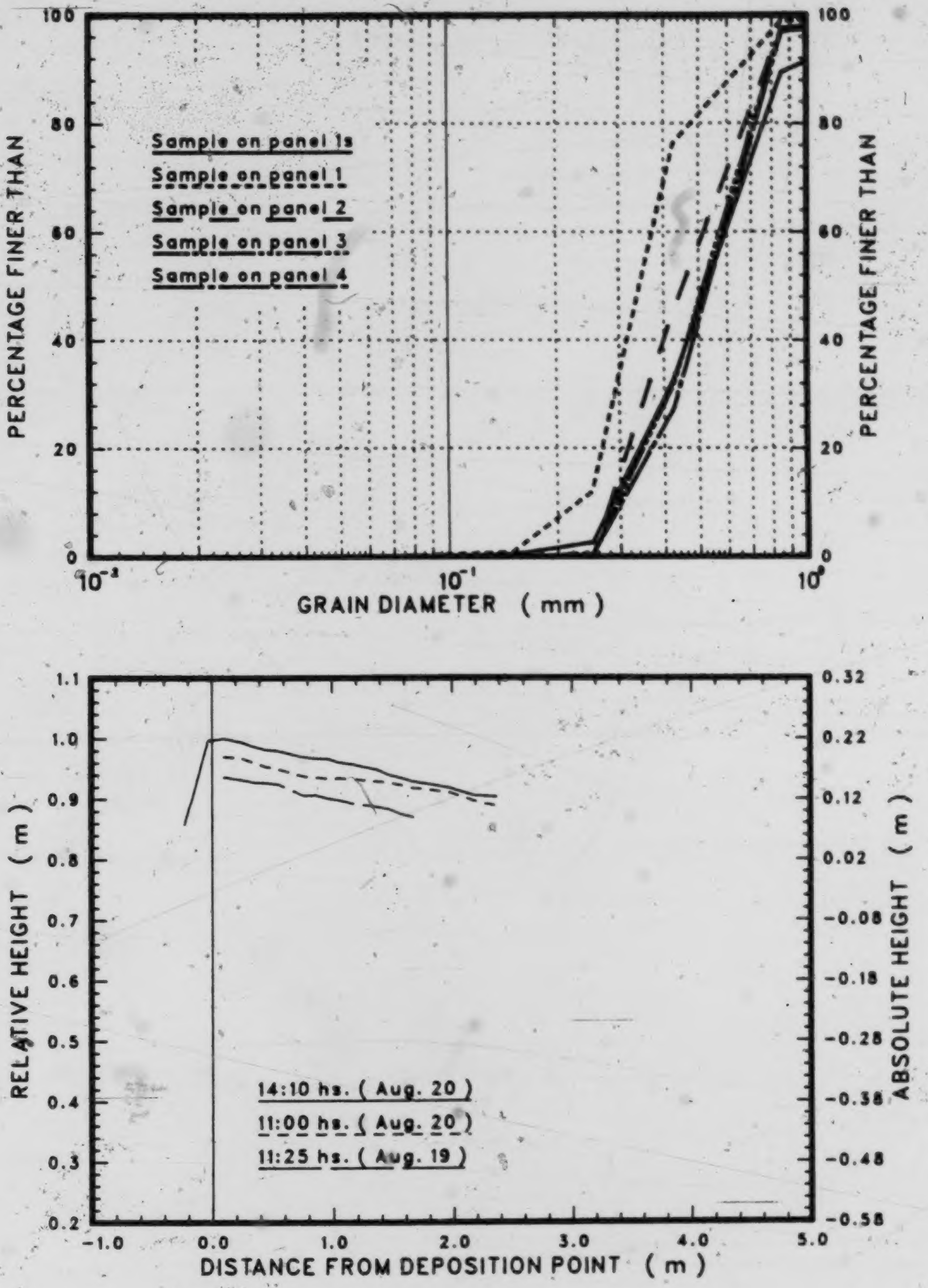


FIG. B.4 – Results of test SS2 (Qt = 15 l/min, Cw = 4 %)

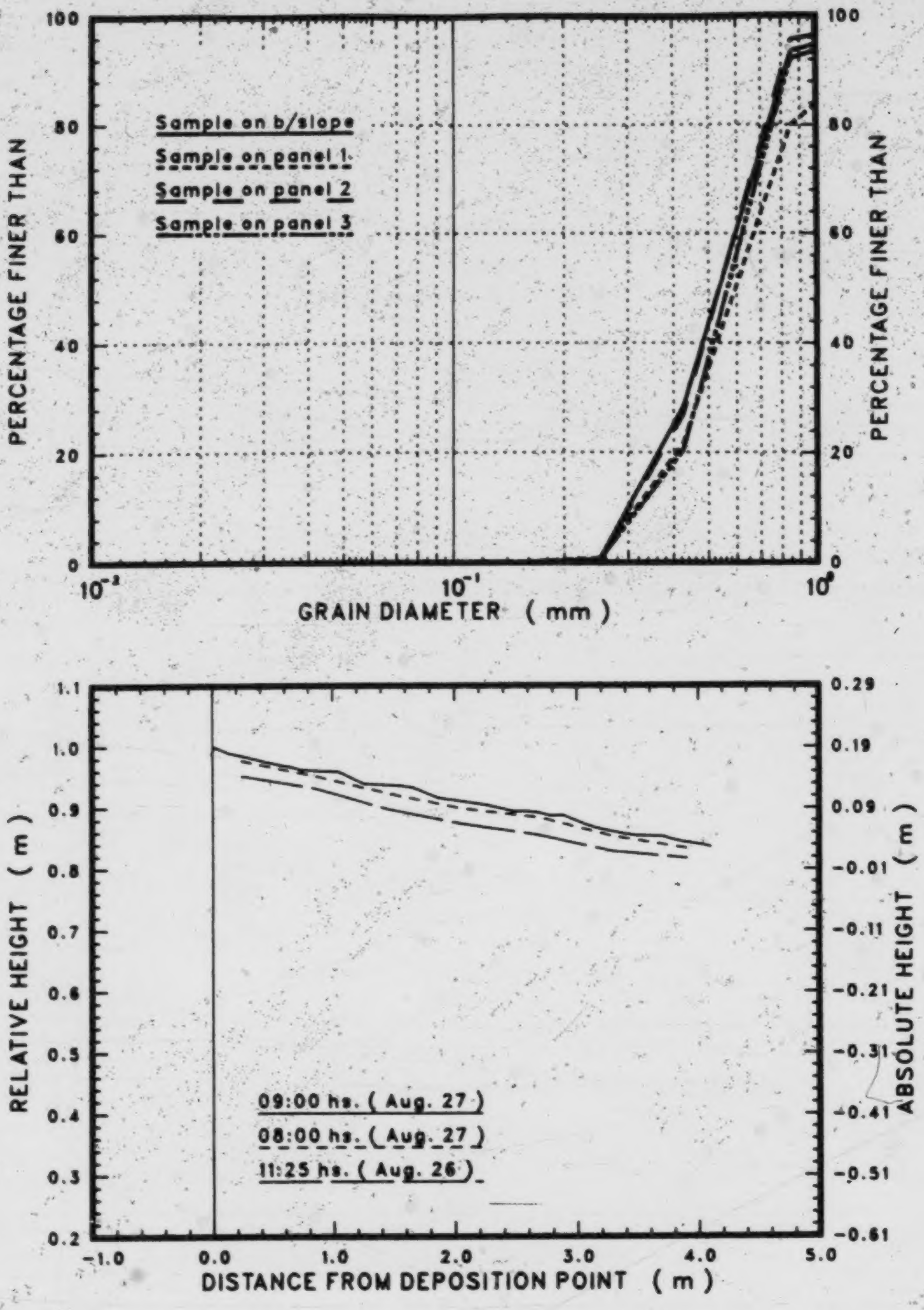


FIG. B.5 – Results of test SS3 (Qt = 15 l/min, Cw = 6 %)

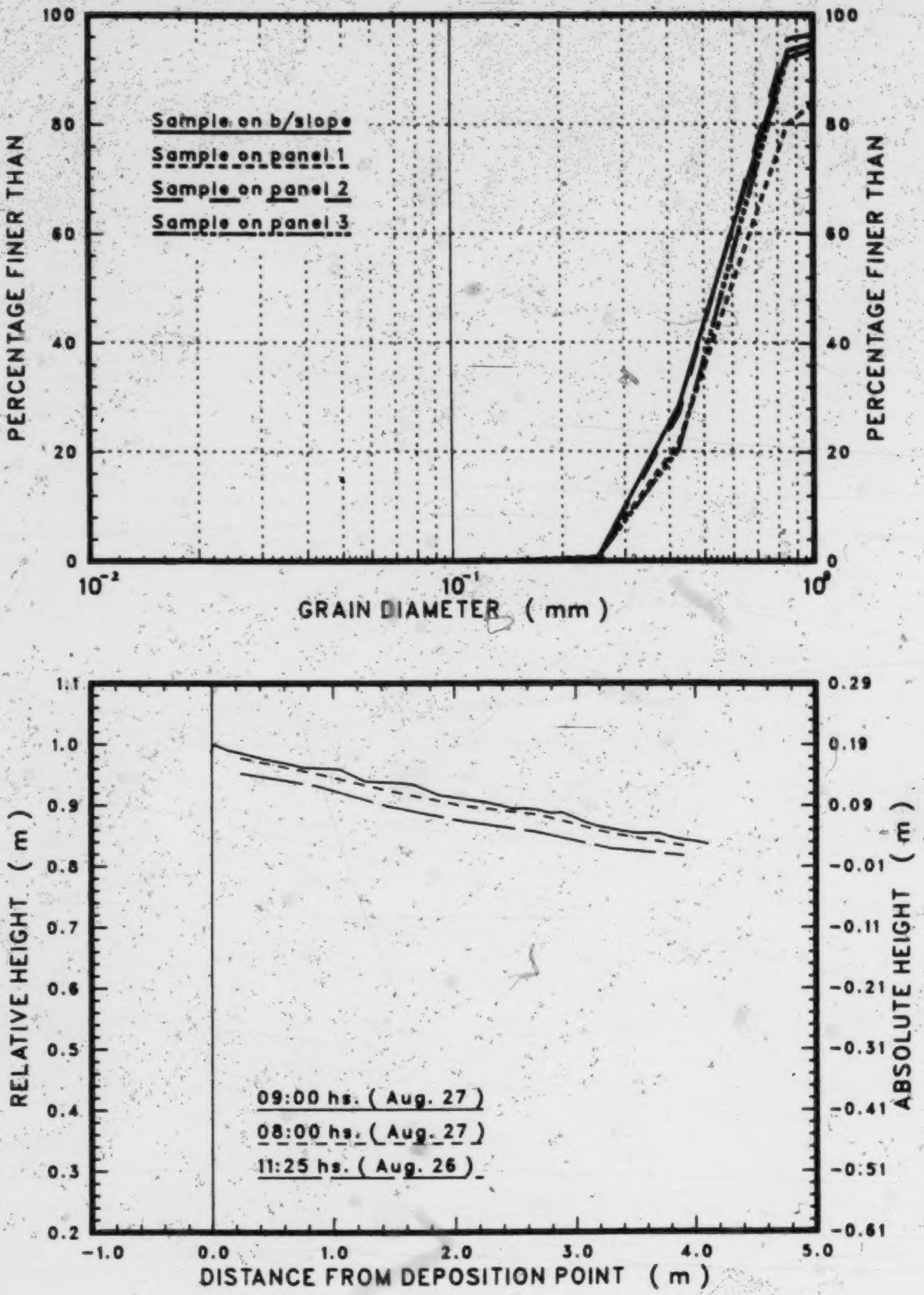
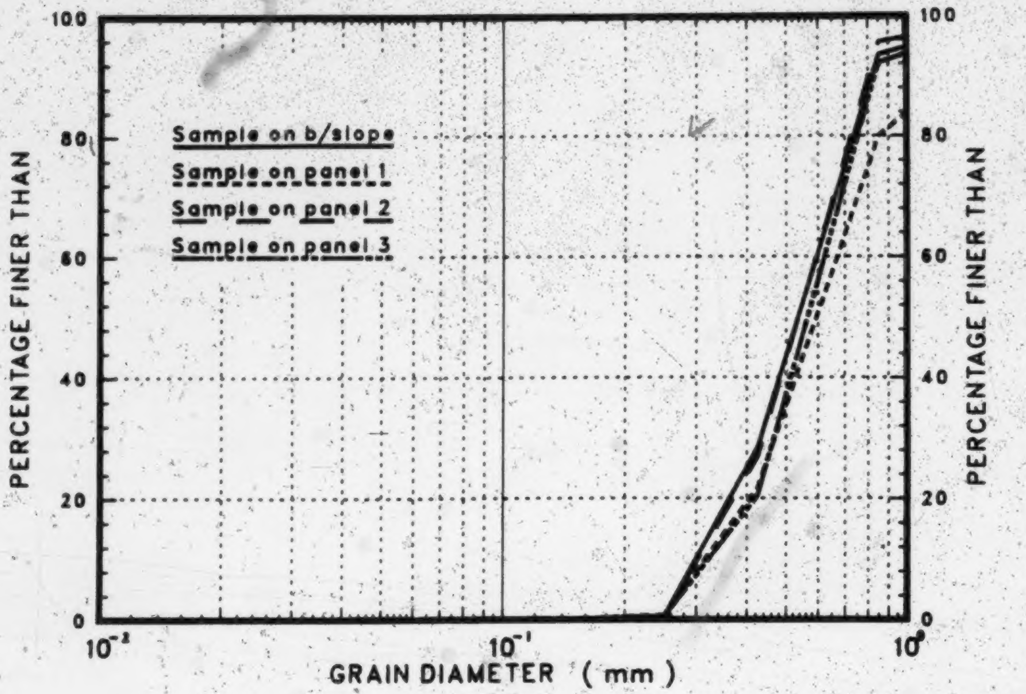


FIG. B.6 – Results of test SS3 (Qt = 15 l/min, Cw = 6 %)



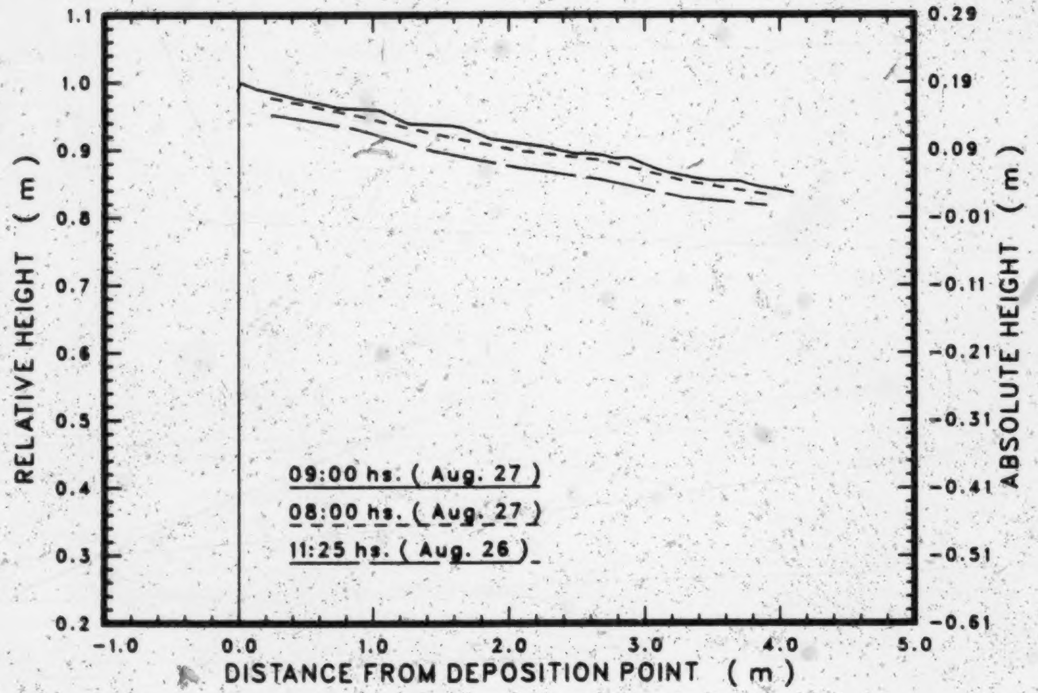
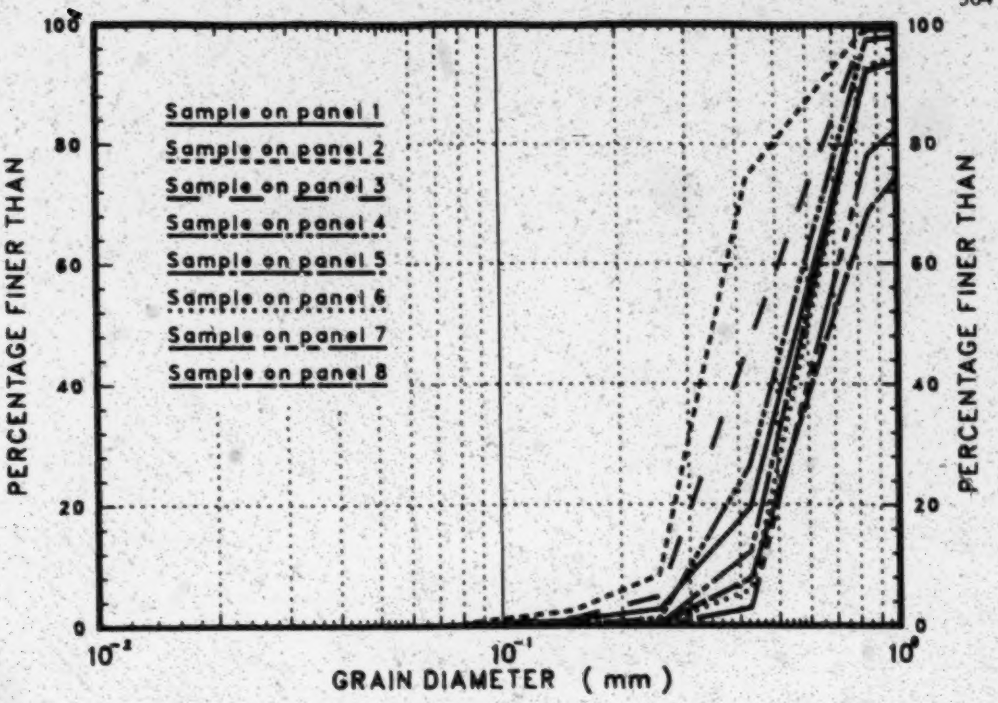


FIG. B.7 – Results of test SS3 (Qt = 15 l/min, Cw = 6 %)



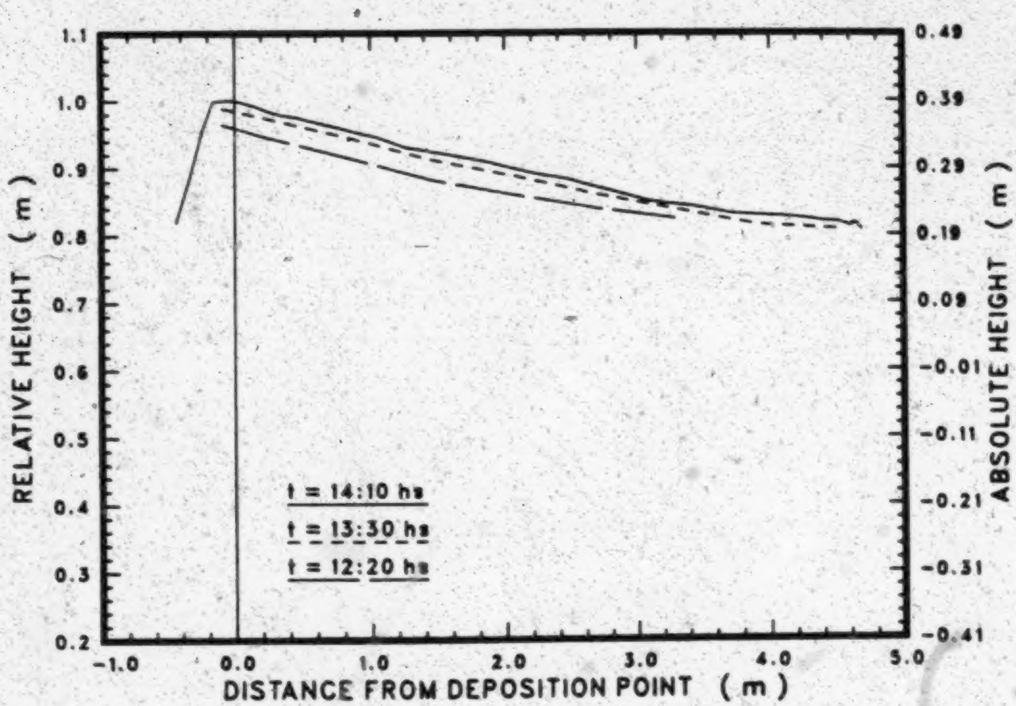
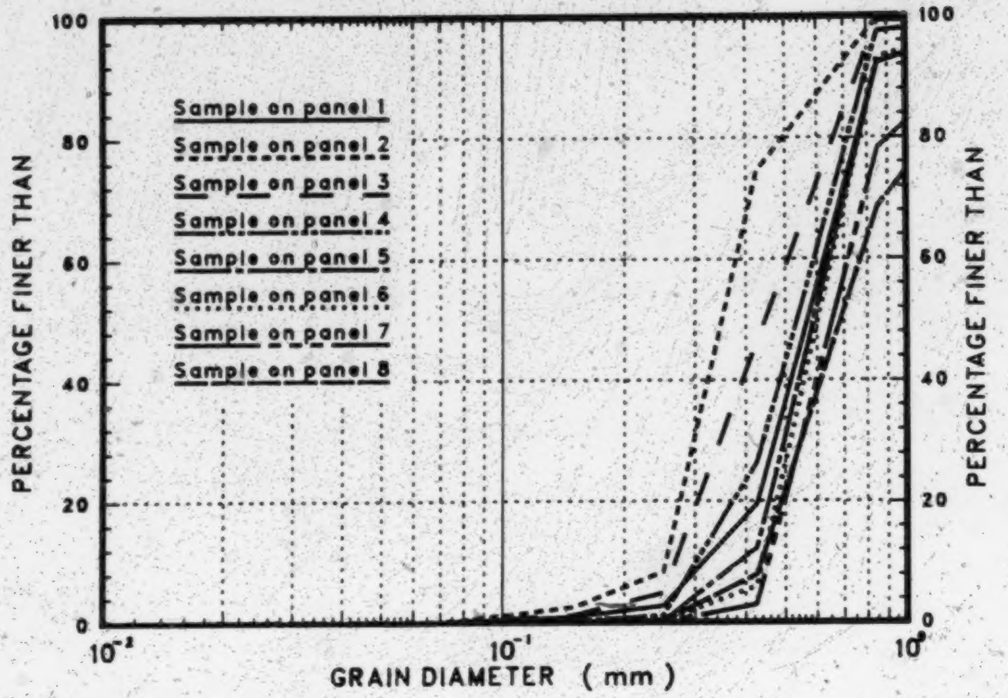


FIG. B.8 – Results of test SS5 (Qt = 15 l/min, Cw = 6 %)



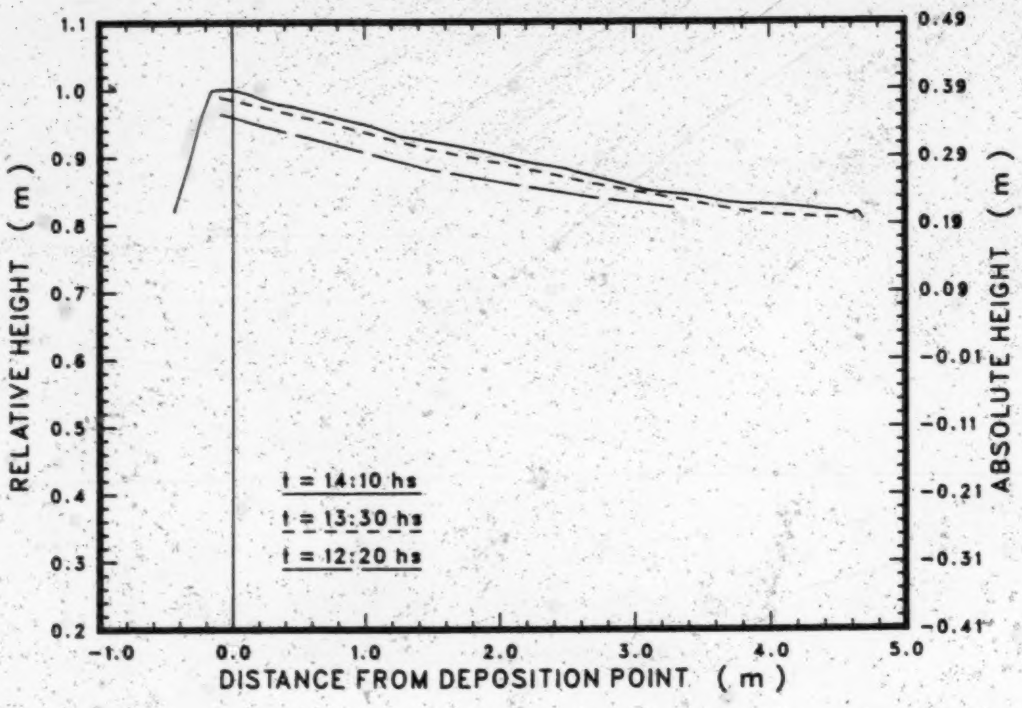
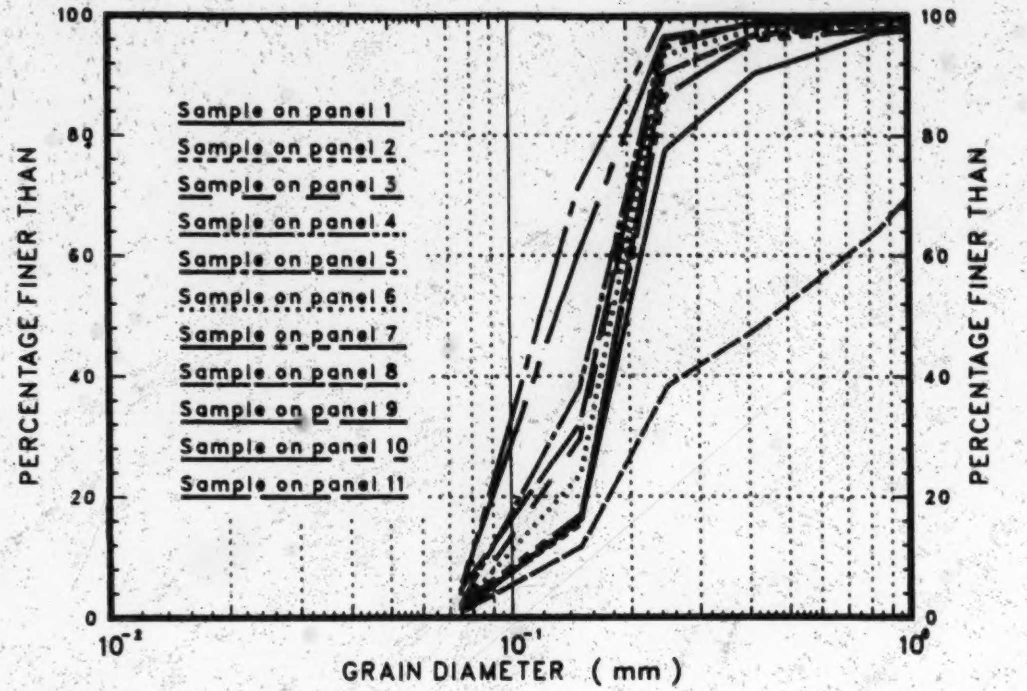


FIG. B.9 – Results of test SS5 (Qt = 15 l/min, Cw = 6 %)



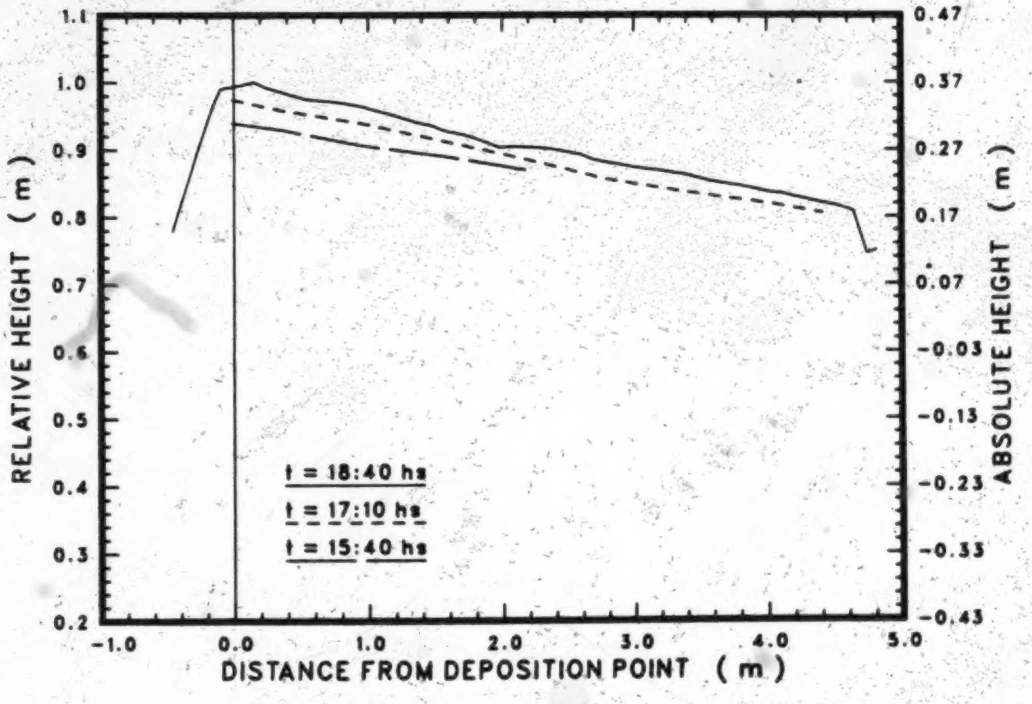


FIG. B.10 – Results of test TS1 (Qt = 15 l/min, Cw = 6 %)

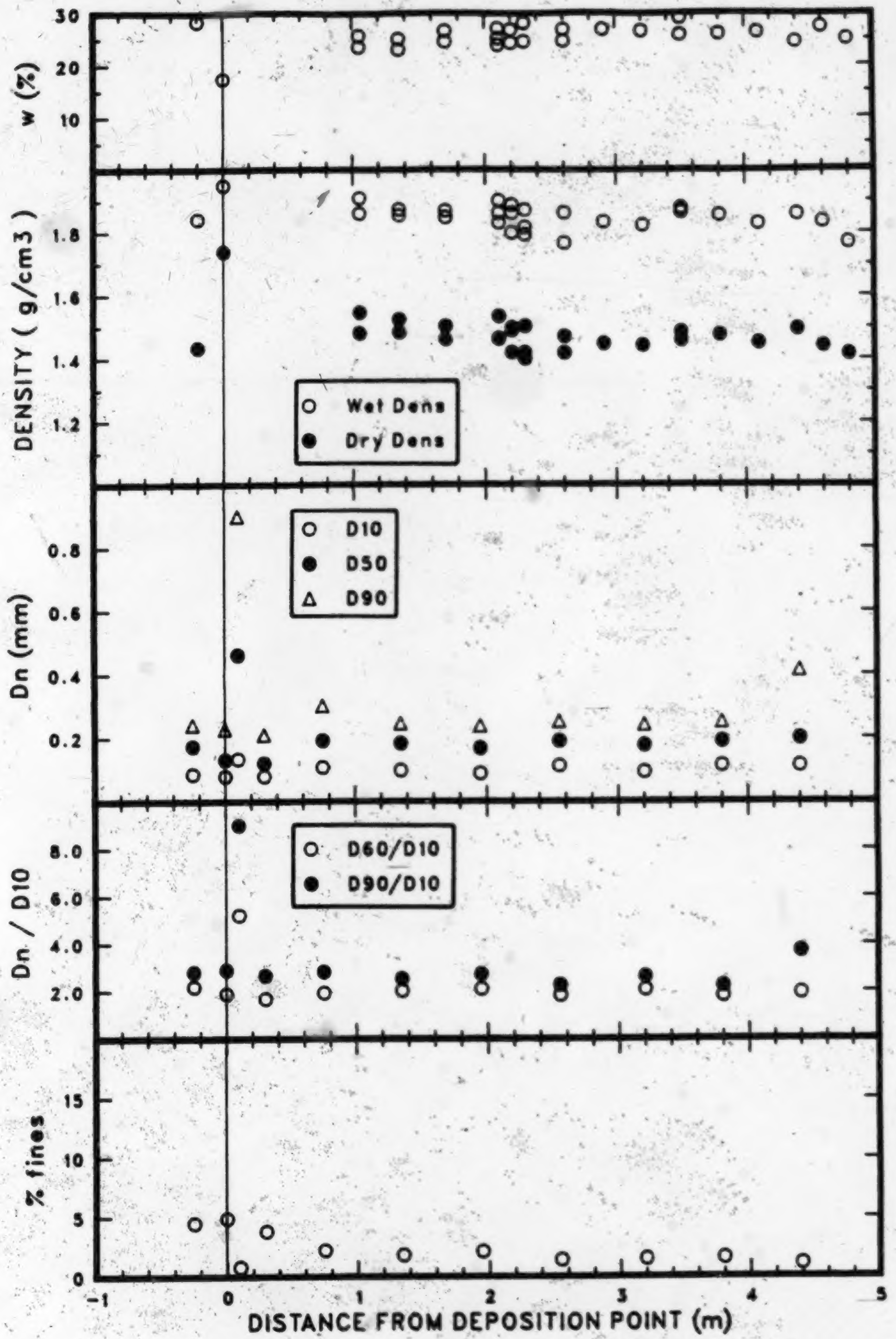


FIG. B.11 – Results of test TS1 (Cont.)

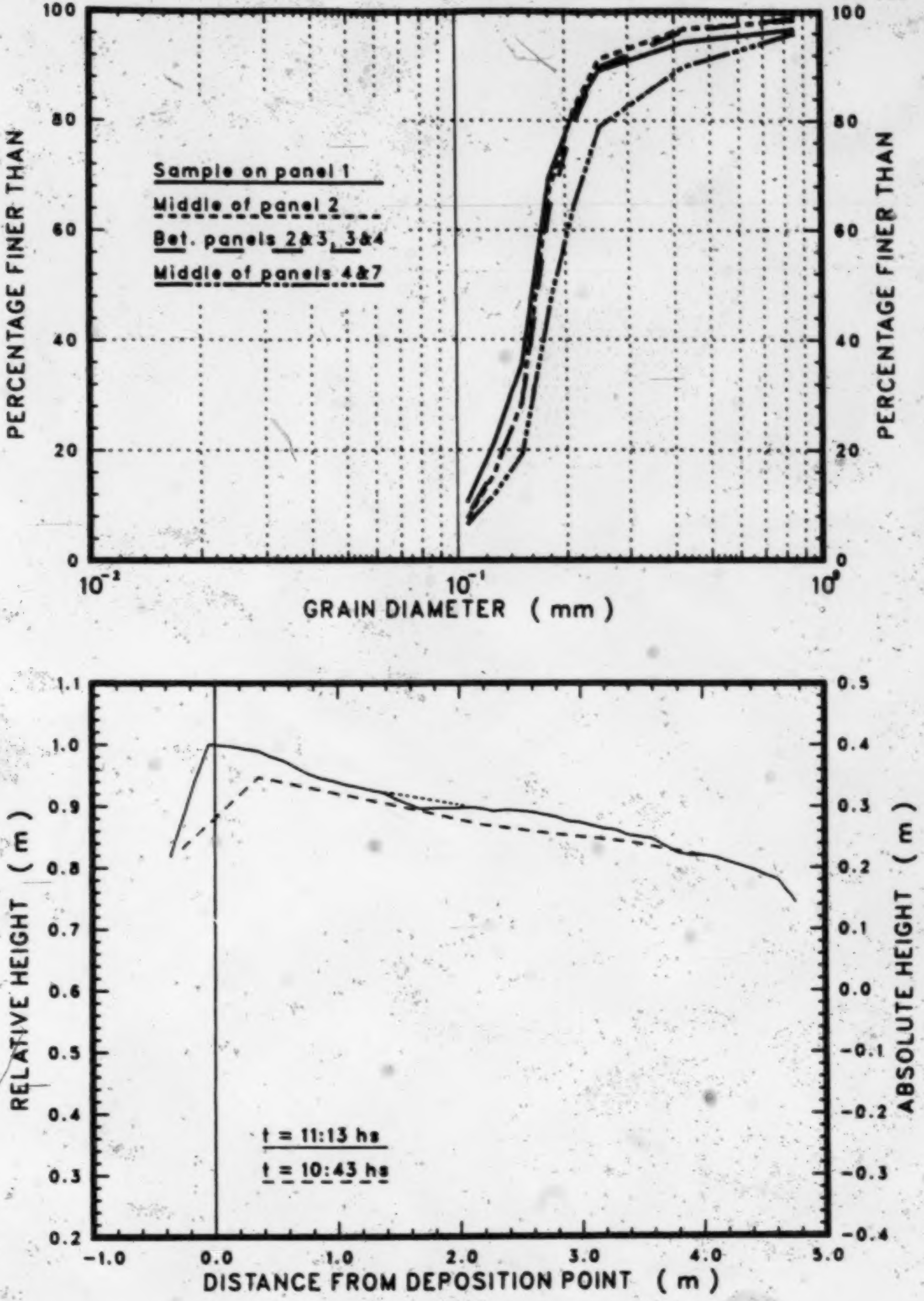


FIG. B.12 – Results of test TS2 (Qt = 12 l/min, Cw = 11%)

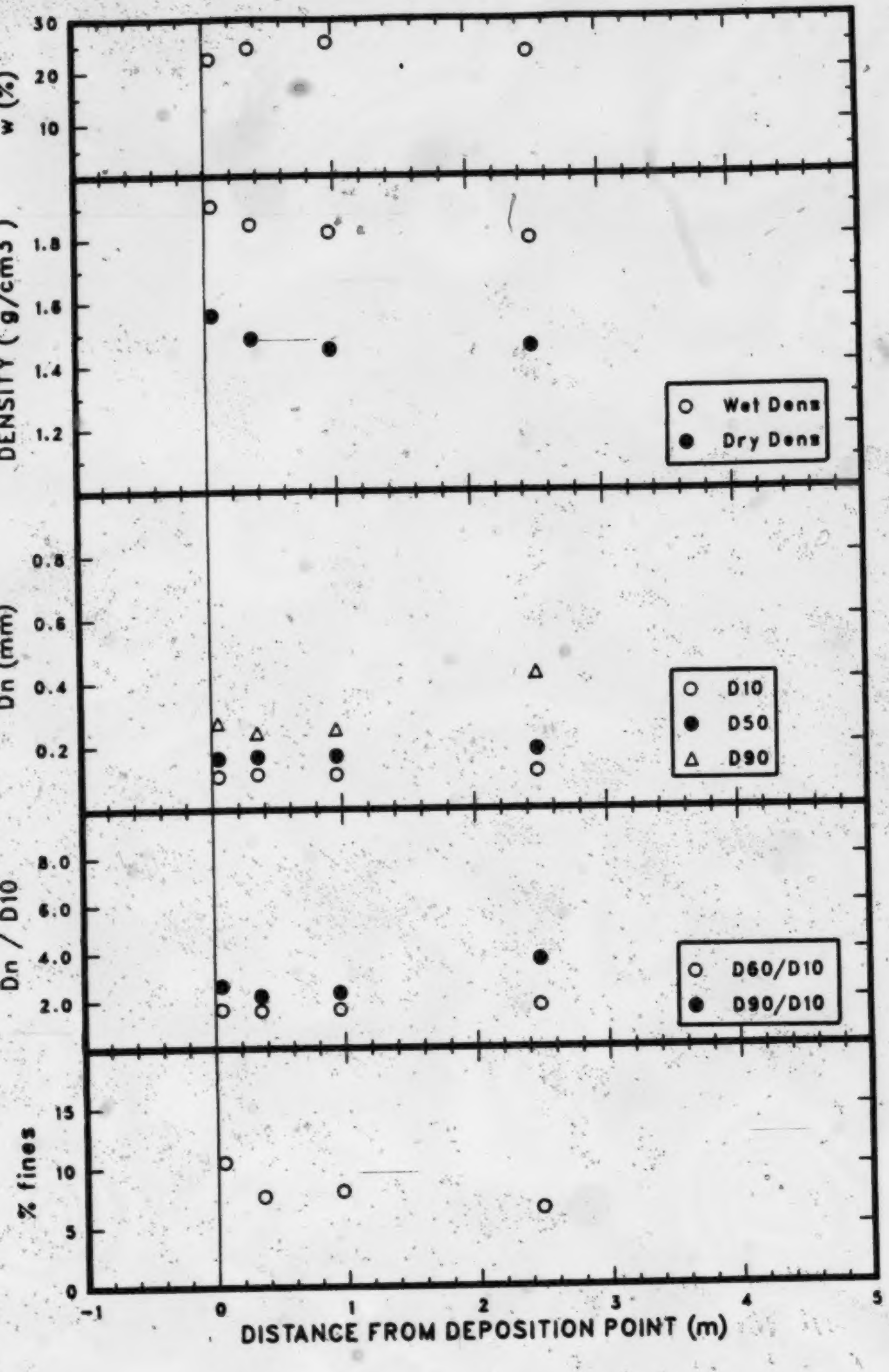


FIG. B.13 – Results of test TS2 (Cont.)

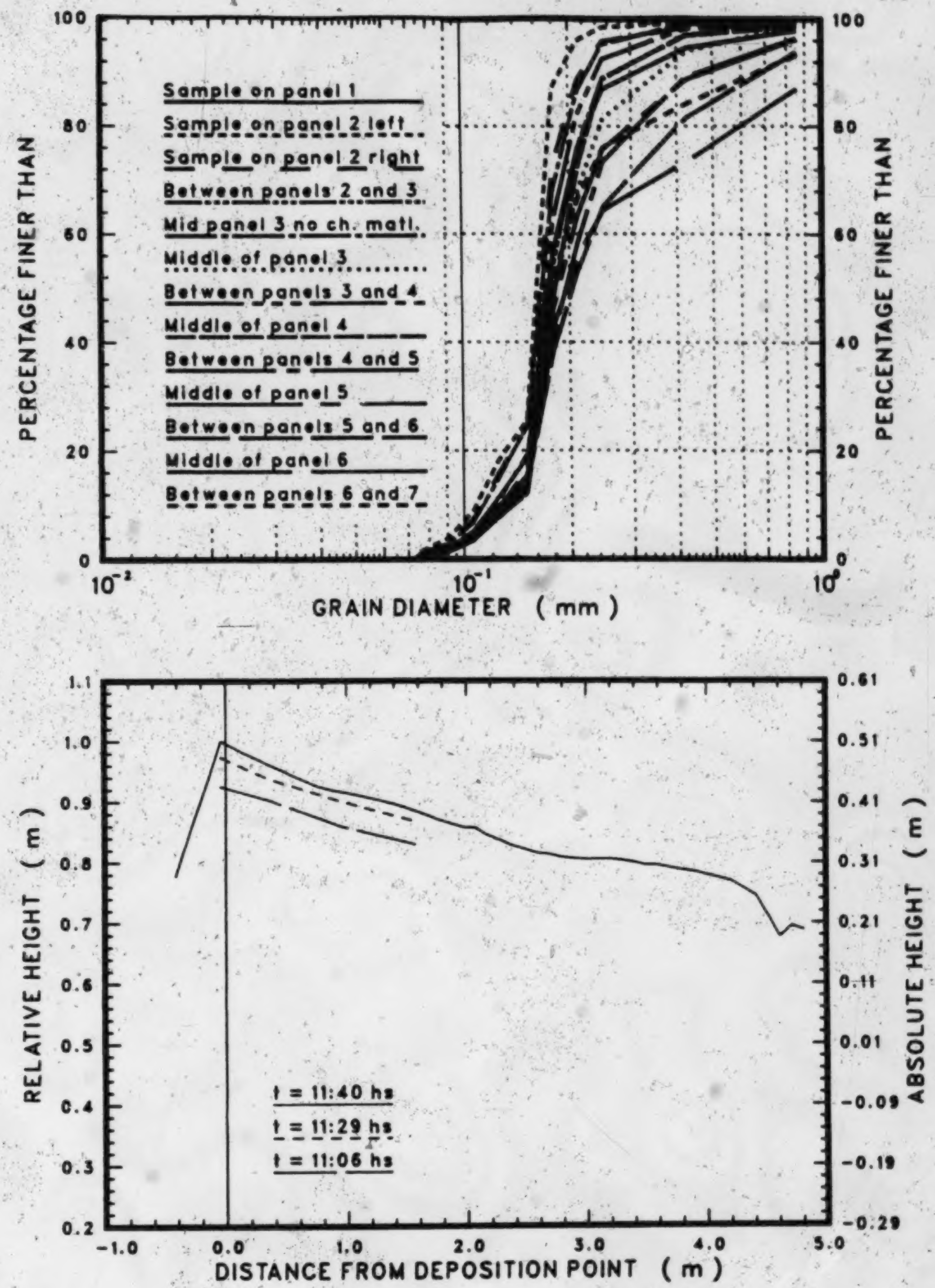


FIG. B.14 – Results of test TS3 (Qt = 12 l/min, Cw = 20 %)

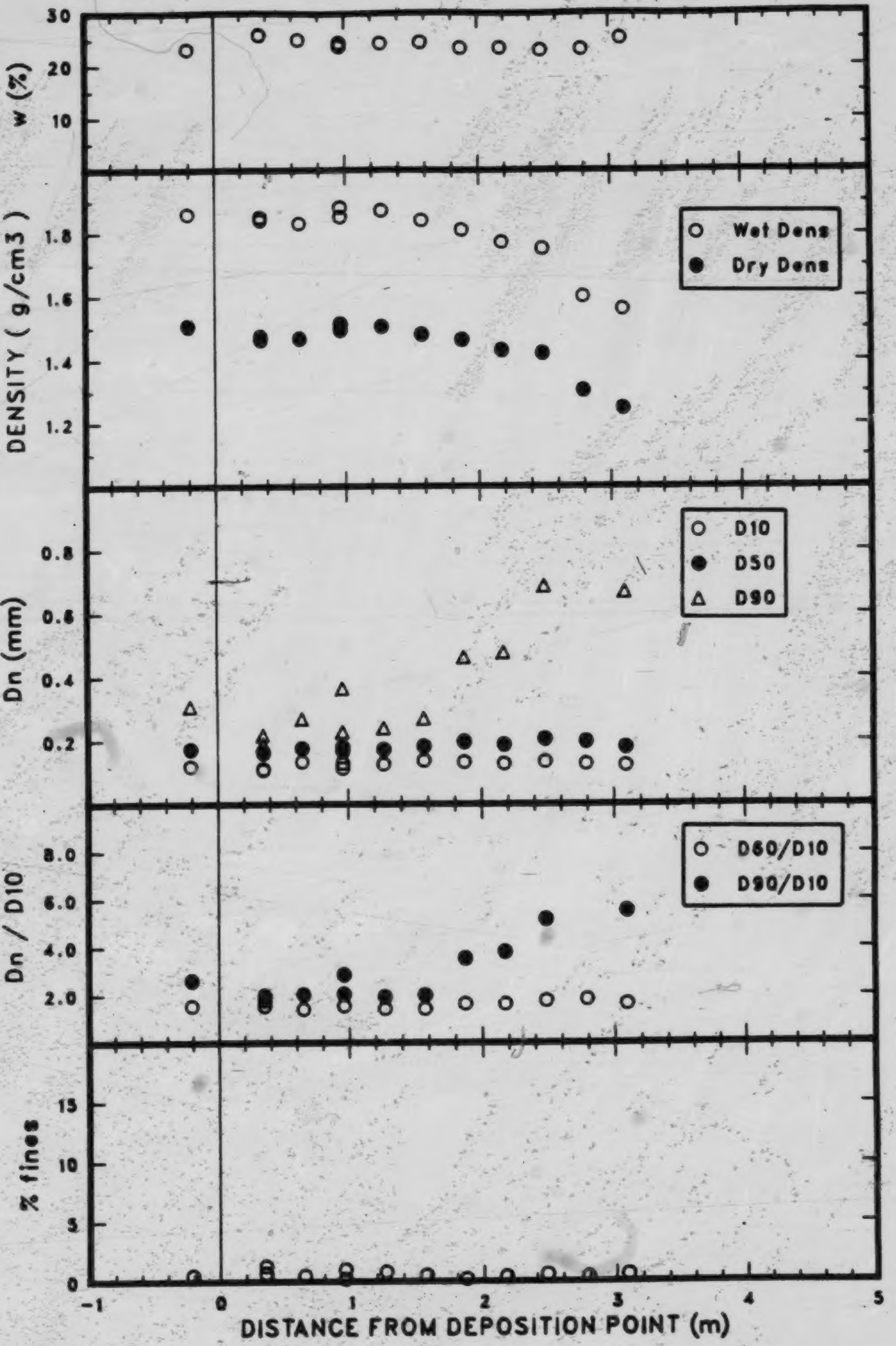


FIG. B.15 – Results of test TS3 (Cont.)

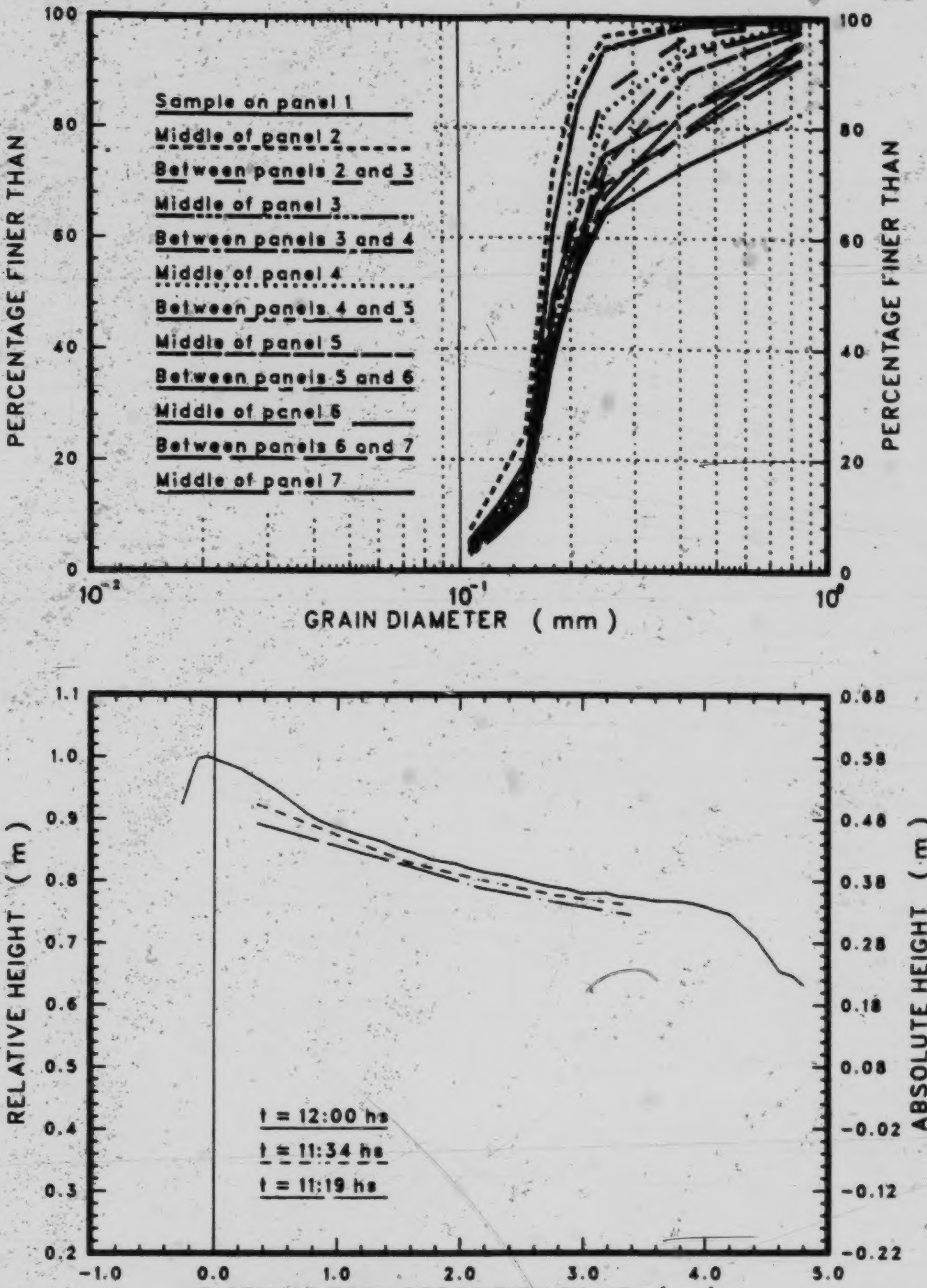


FIG. B.16 – Results of test TS4 (Qt = 12 l/min, Cw = 20 %)

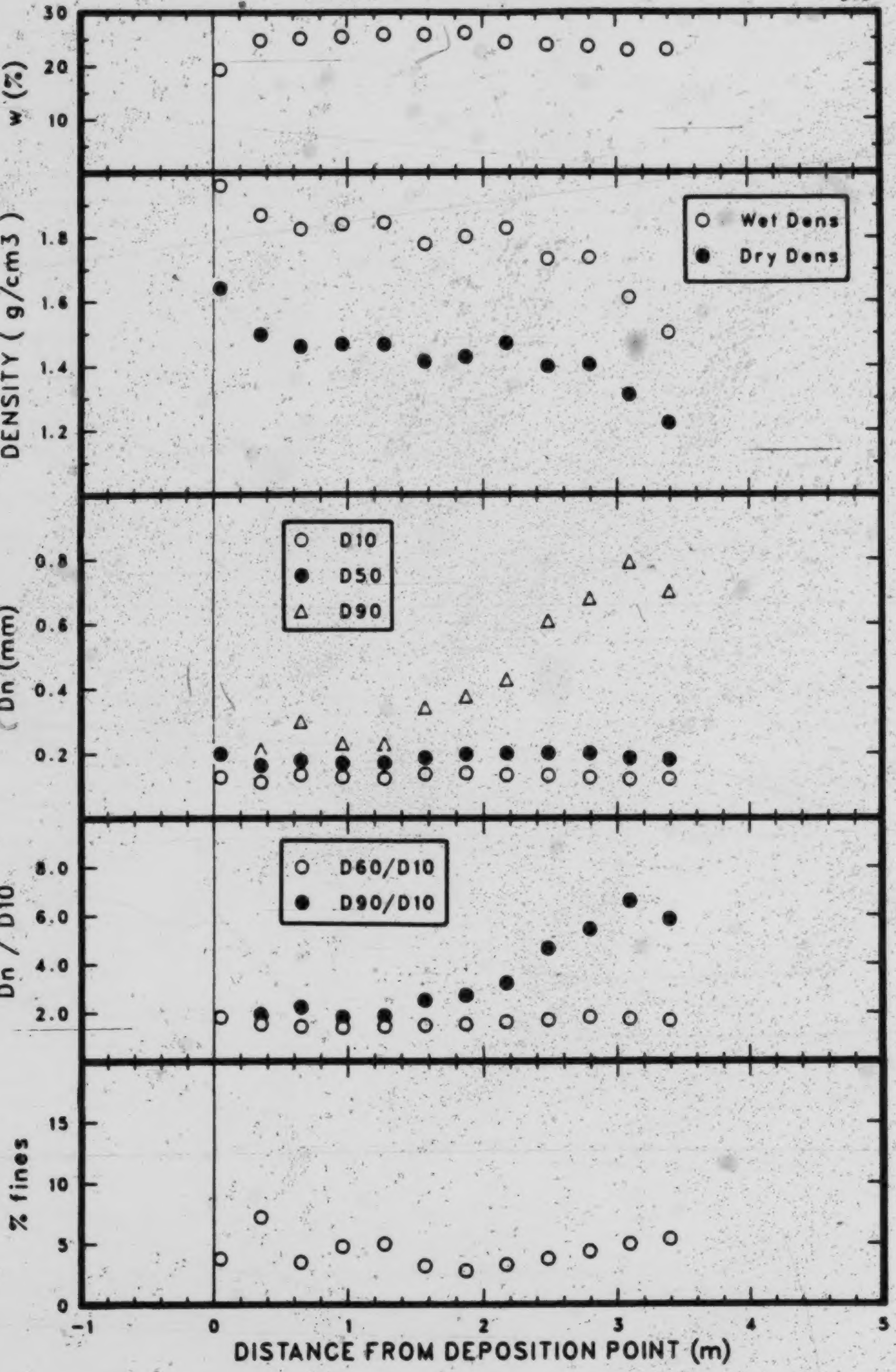


FIG. B.17 – Results of test TS4 (Cont.)

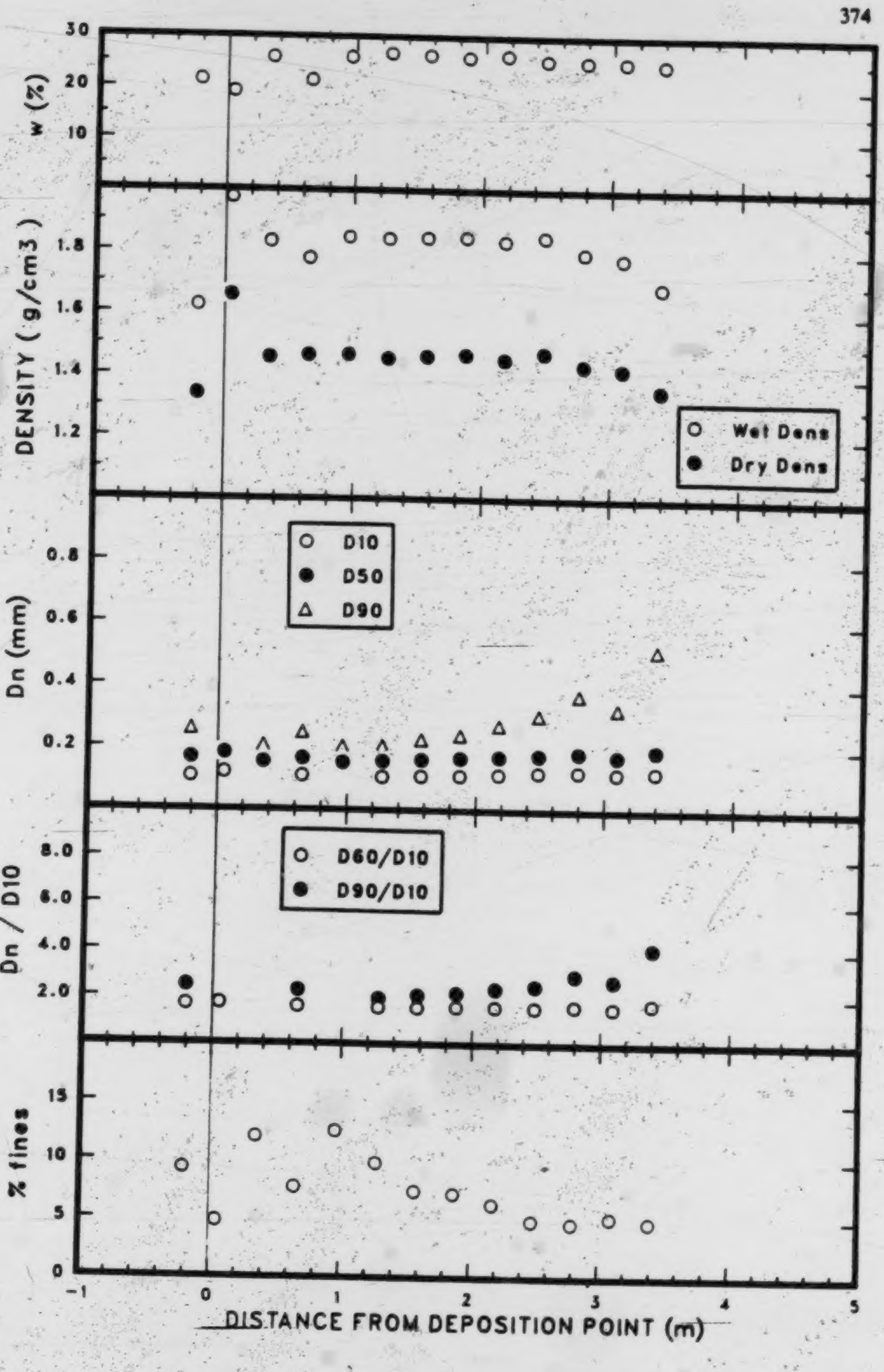


FIG. B.18 – Results of test TS5 (Cont.)

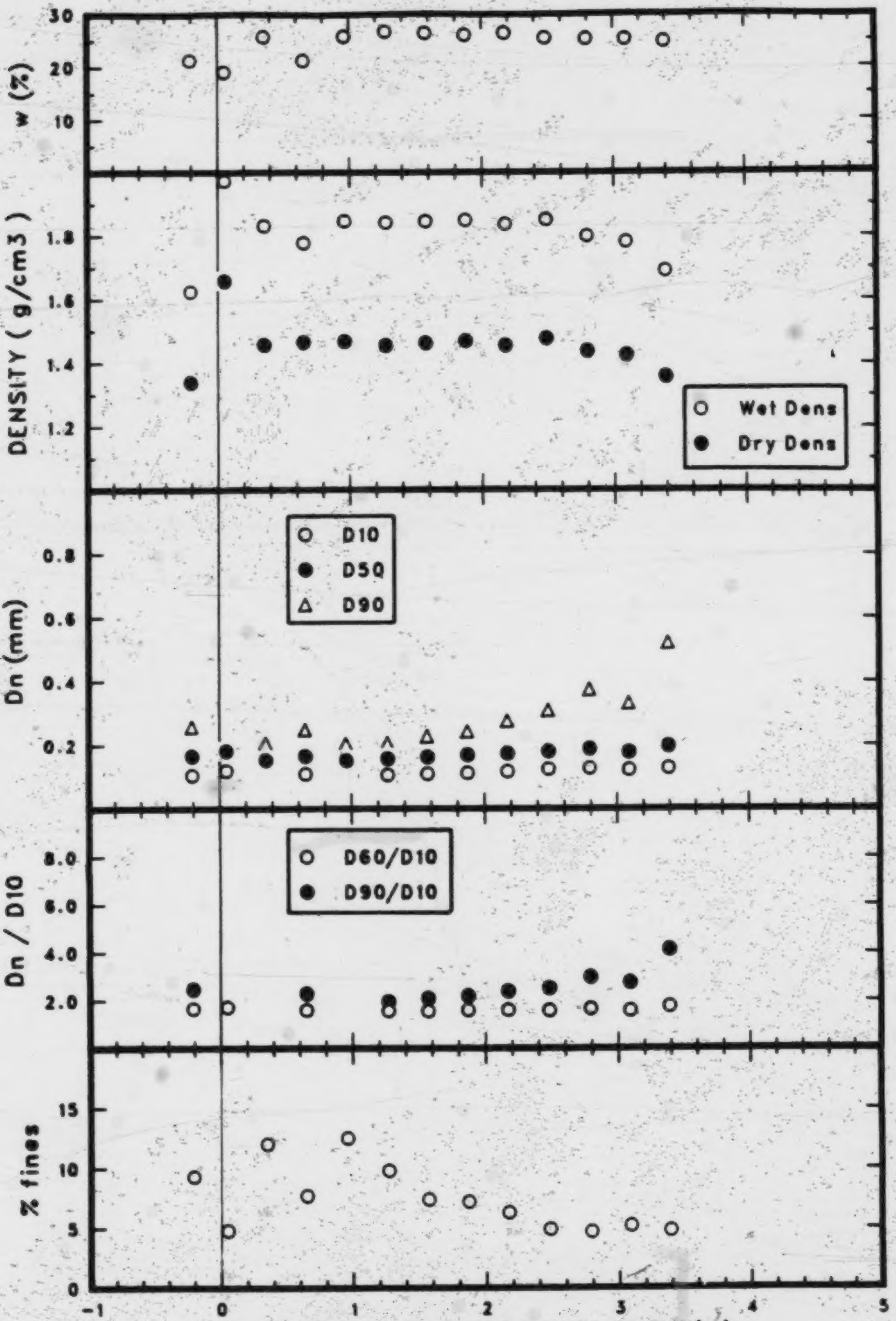


FIG. B.19 – Results of test TS5 (Cont.)

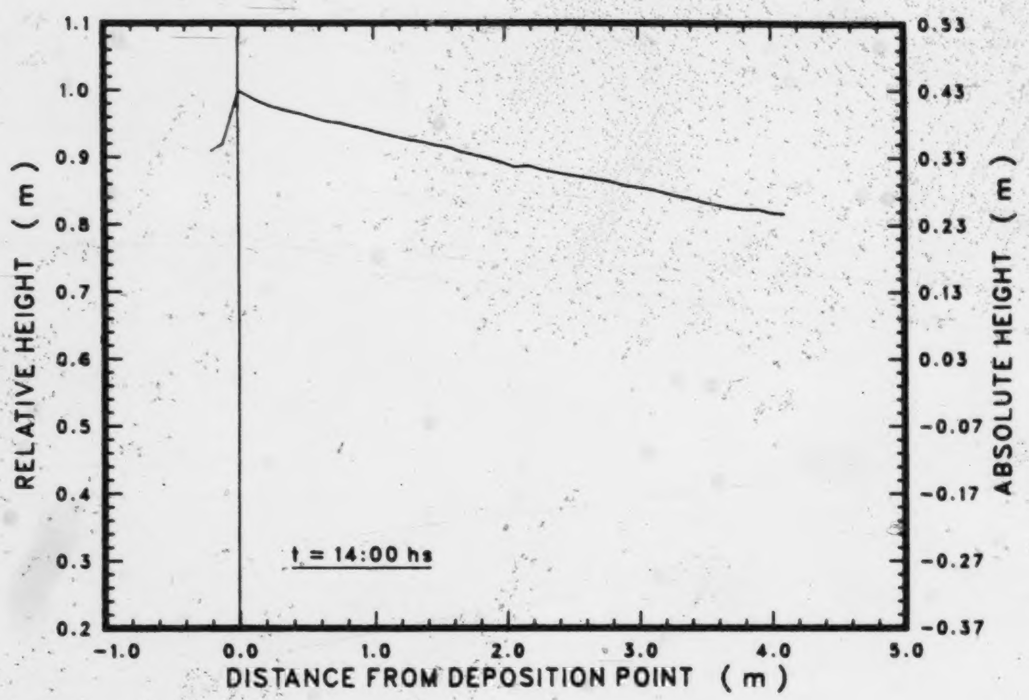


FIG. B.20 – Results of test TS6 (Qt = 5 l/min, Cw = 2 %)

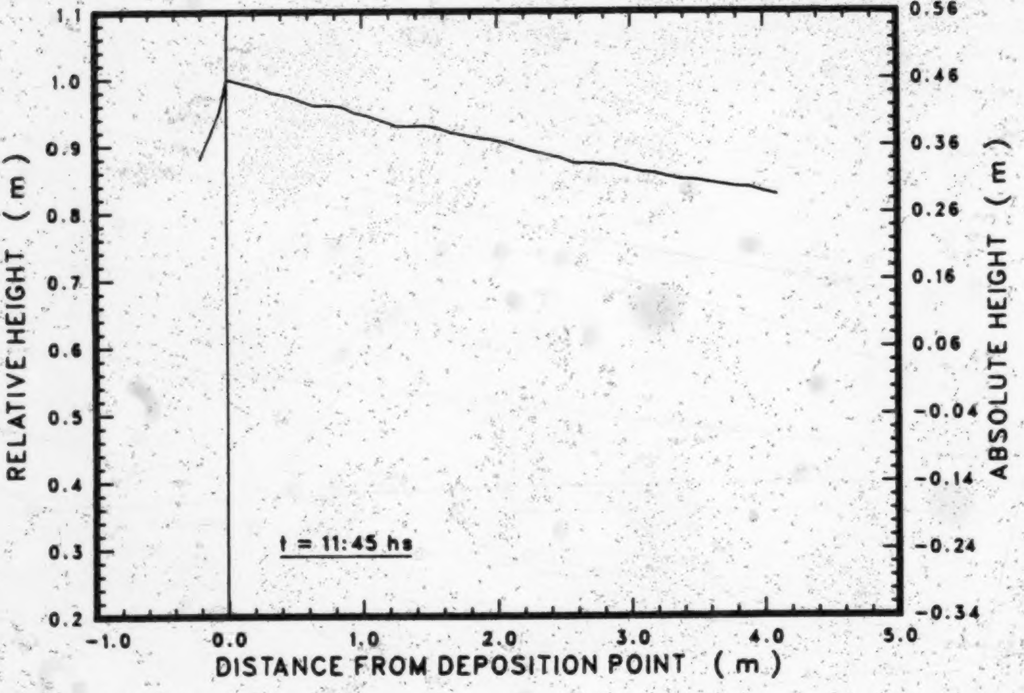


FIG. B.21 – Results of test TS7 (Qt = 5 l/min, Cw = 2 %)

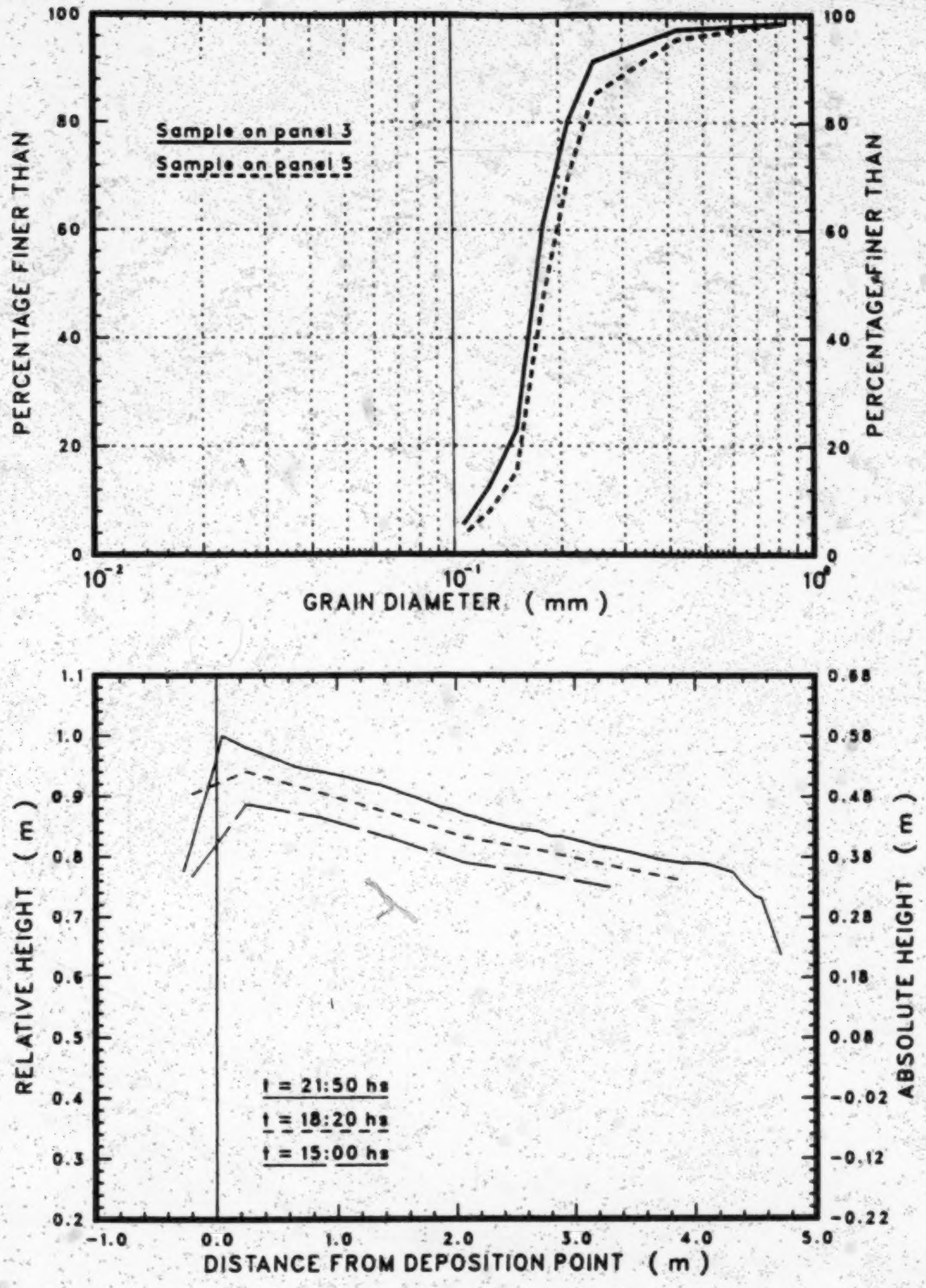


FIG. B.22 – Results of test TS8 (Qt = 5 l/min, Cw = 6 %)

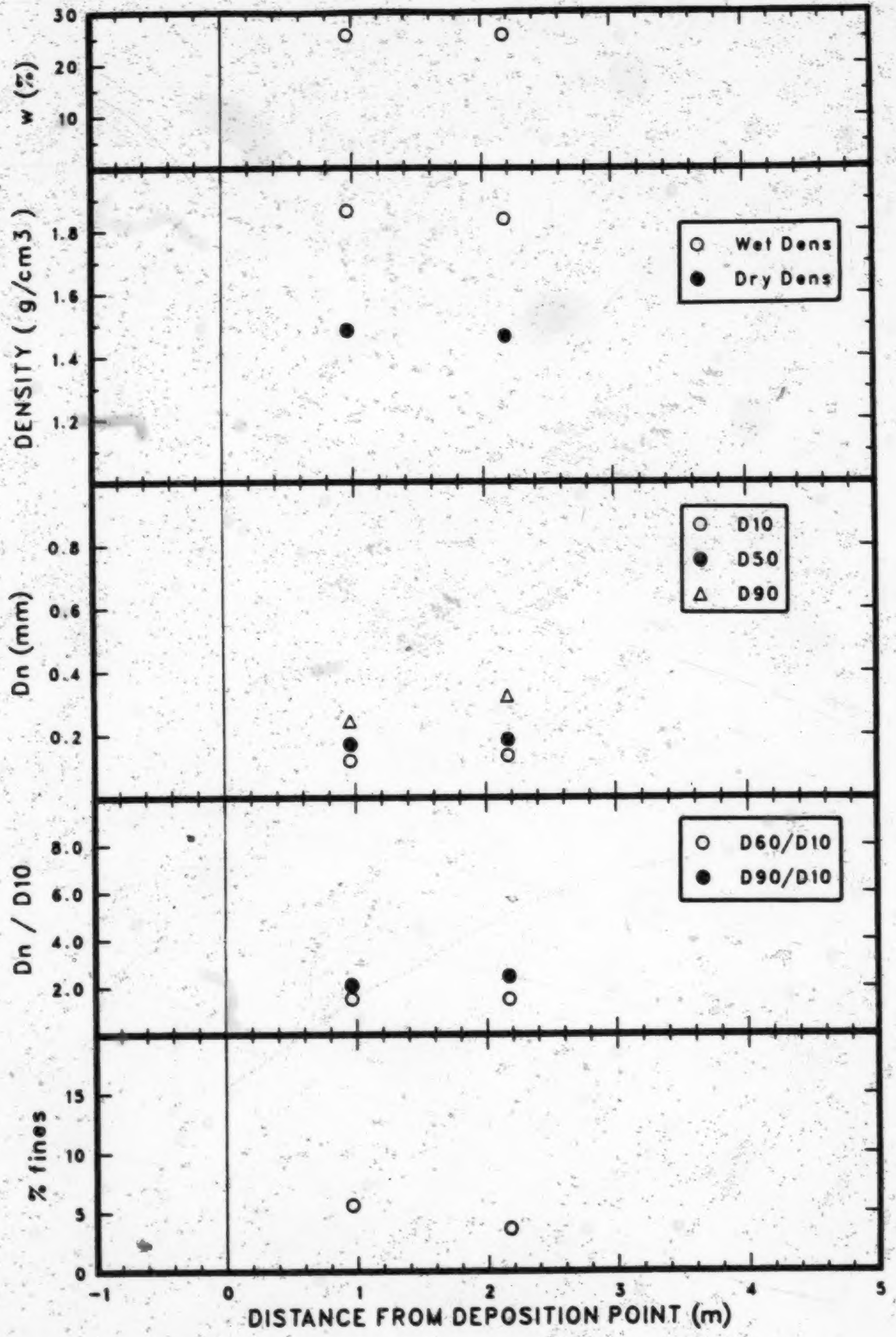


FIG. B.23 – Results of test TS8 (Cont.)

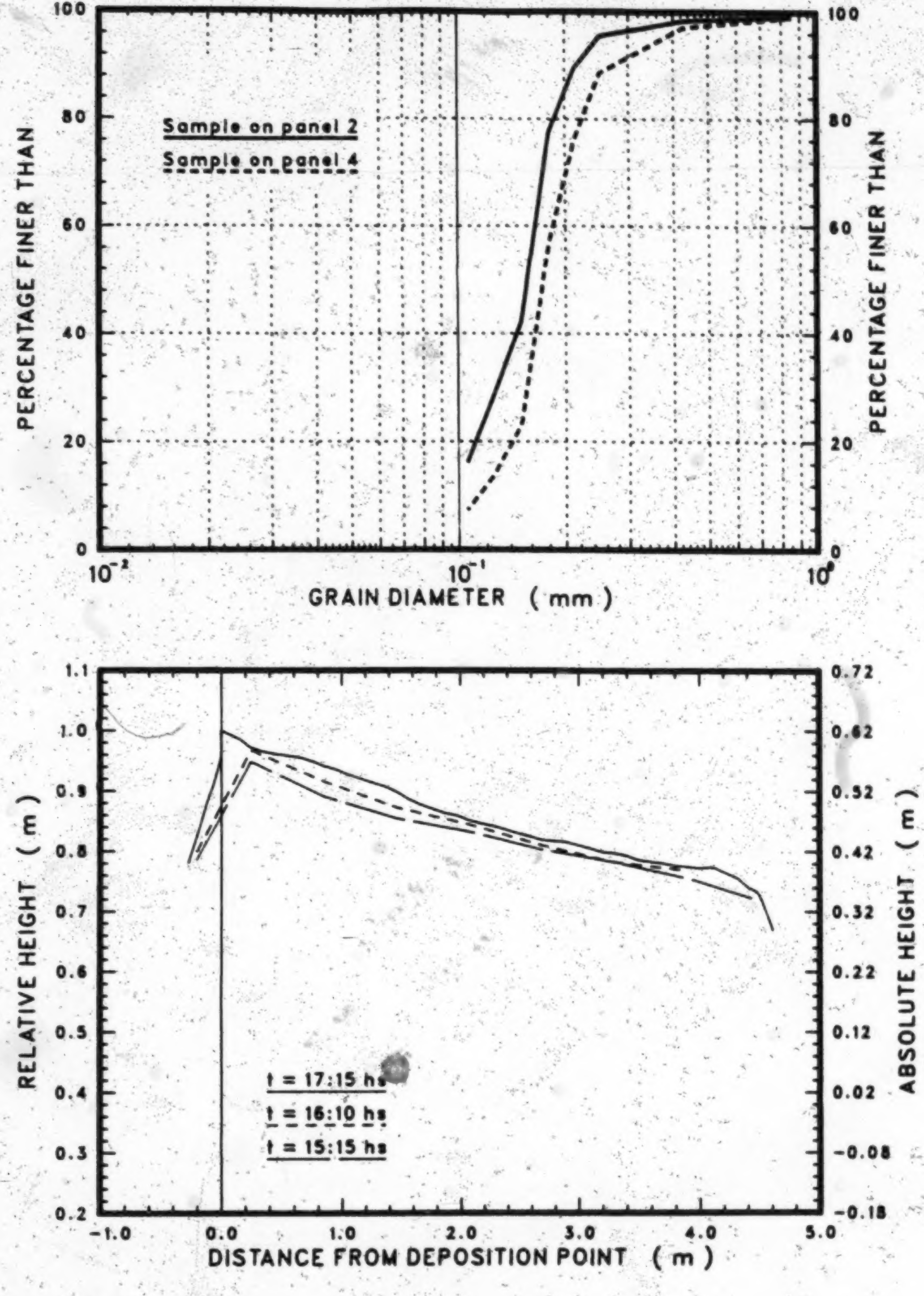


FIG. B.24 – Results of test TS9 (Qt = 5 l/min, Cw = 10 %)

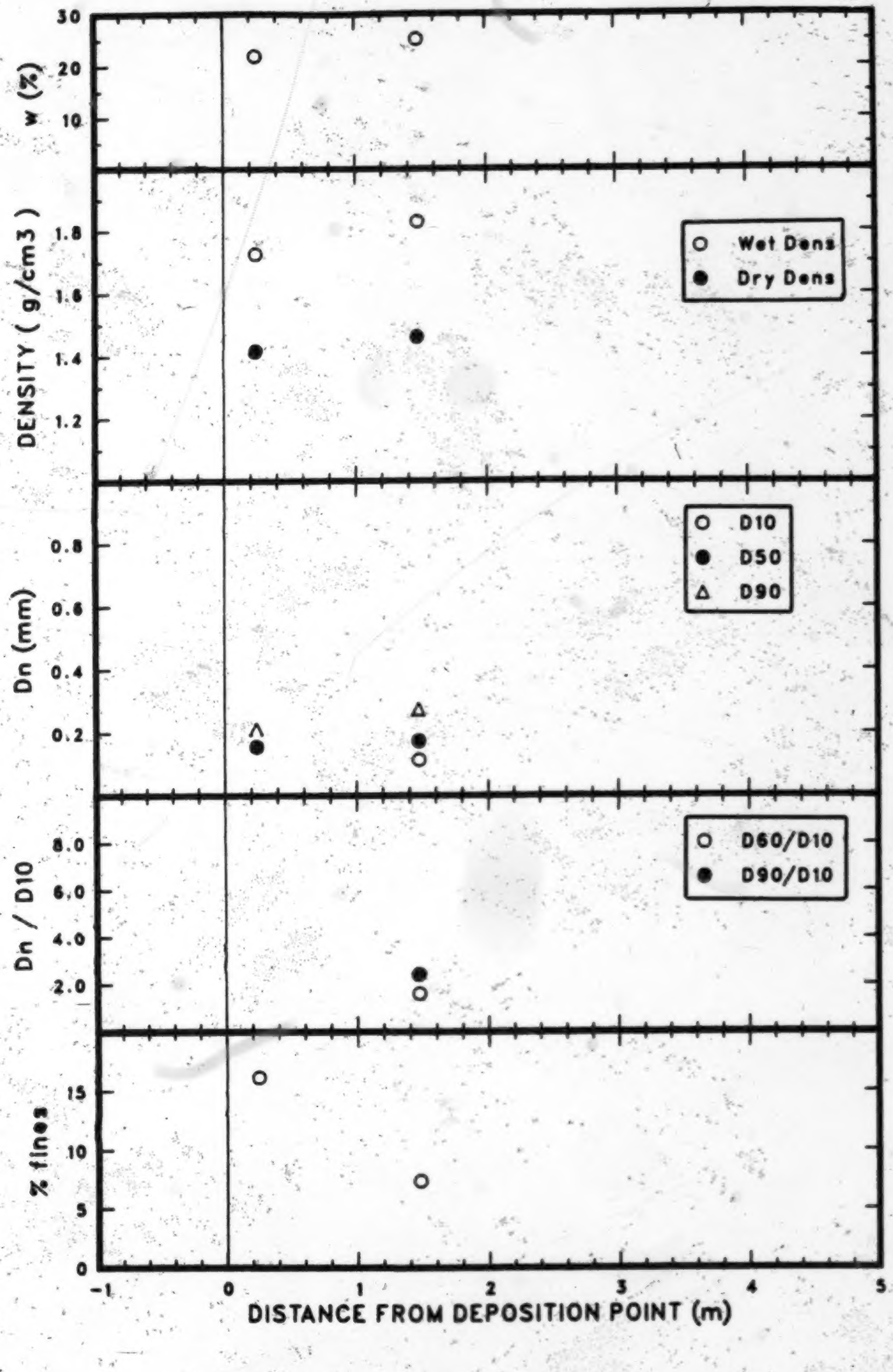
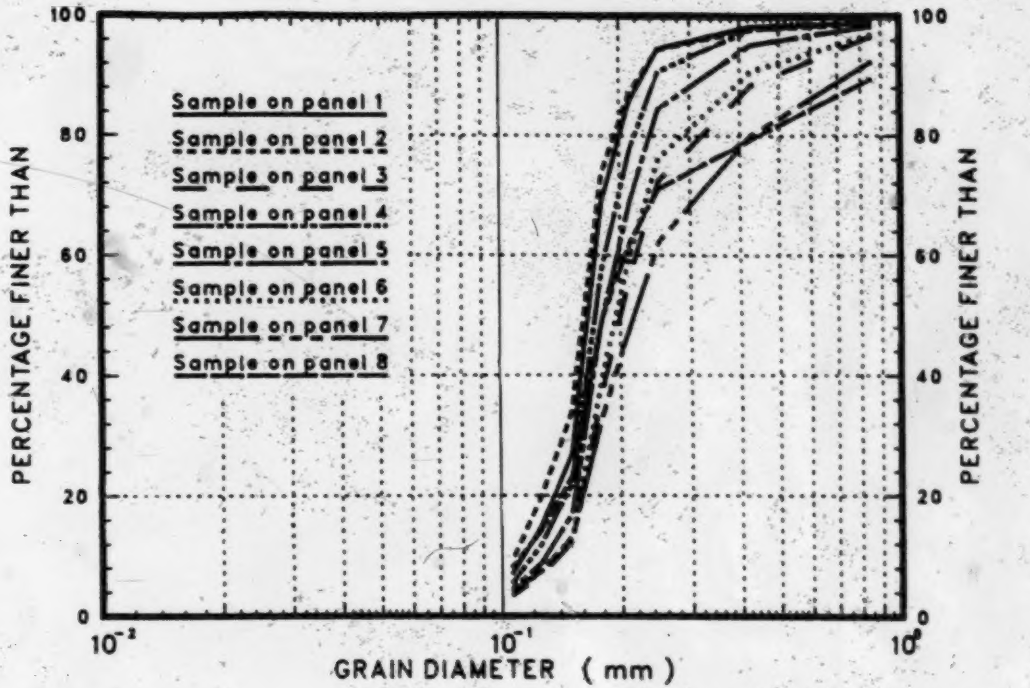


FIG. B.25 – Results of test TS9 (Cont.)



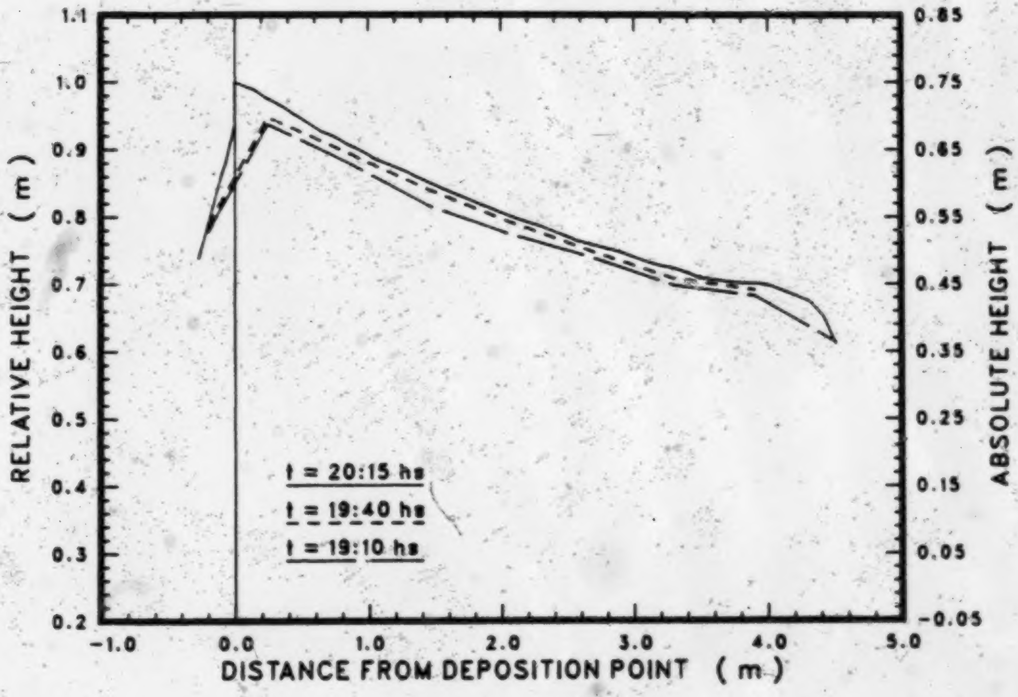


FIG. B.26 – Results of test TS10 (Qt = 5 l/min, Cw = 15 %)

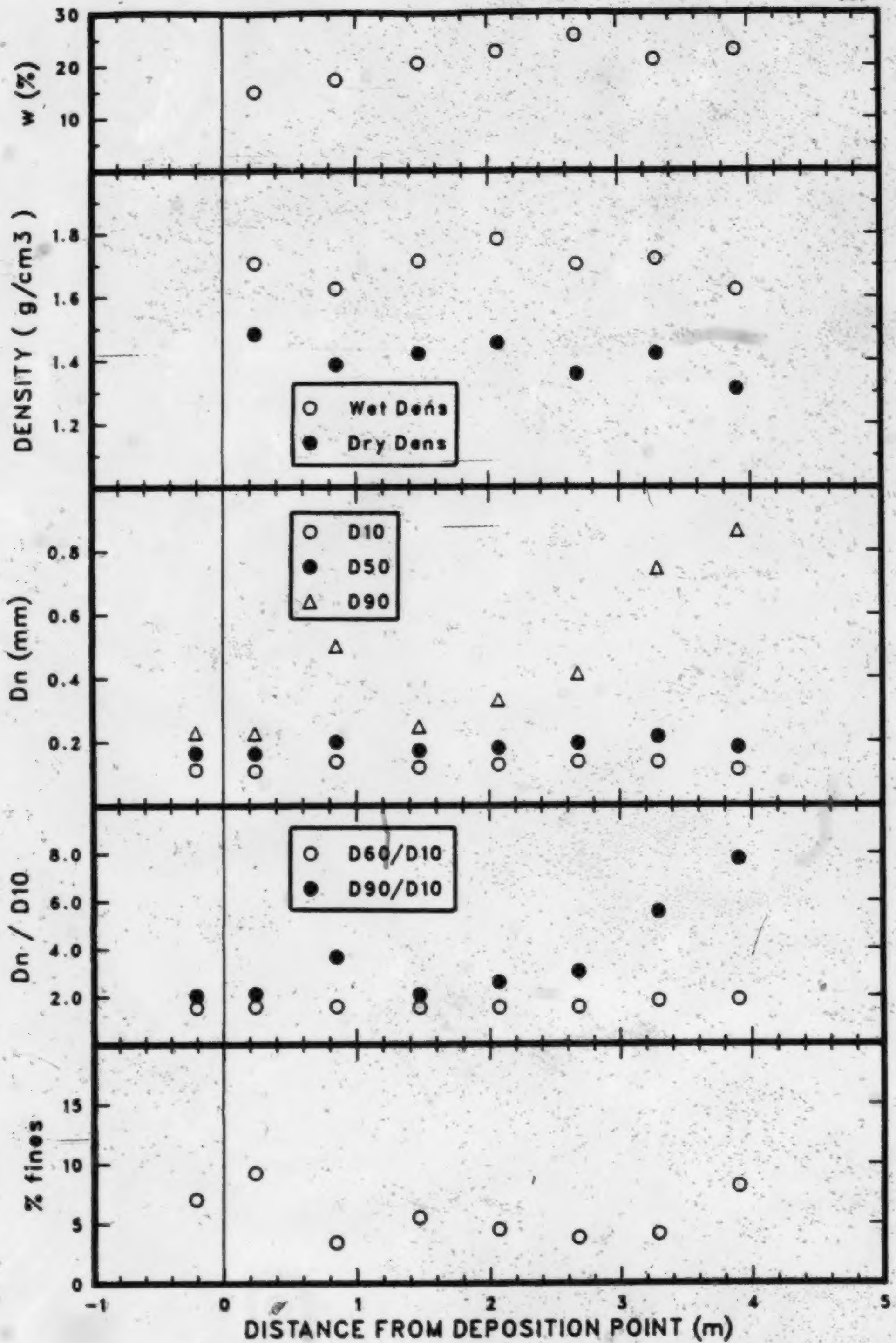
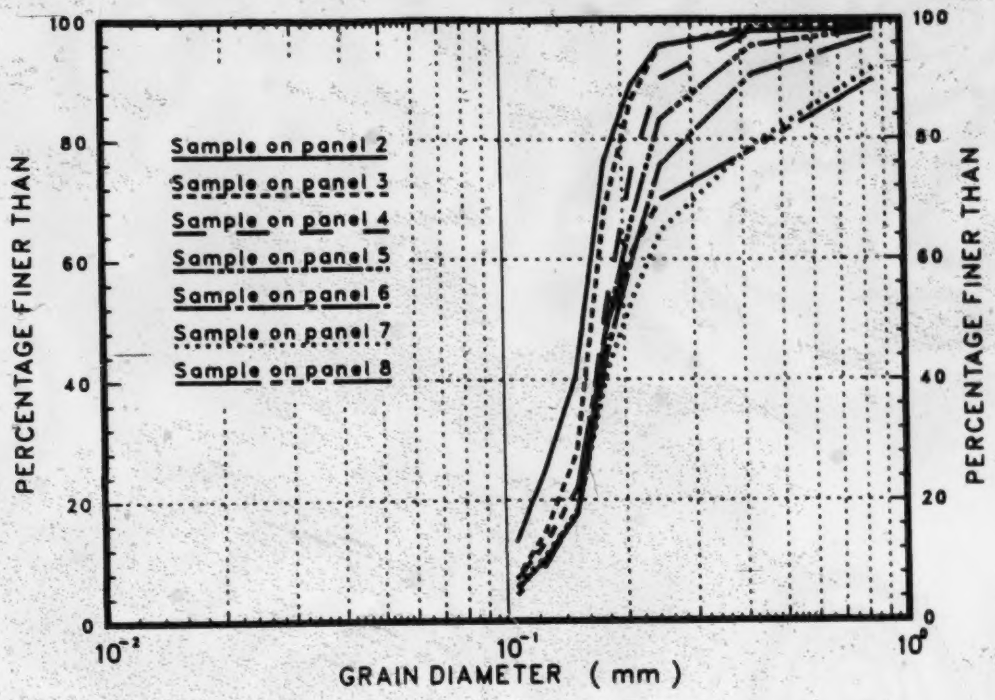


FIG. B.27 – Results of test TS10 (Cont.)



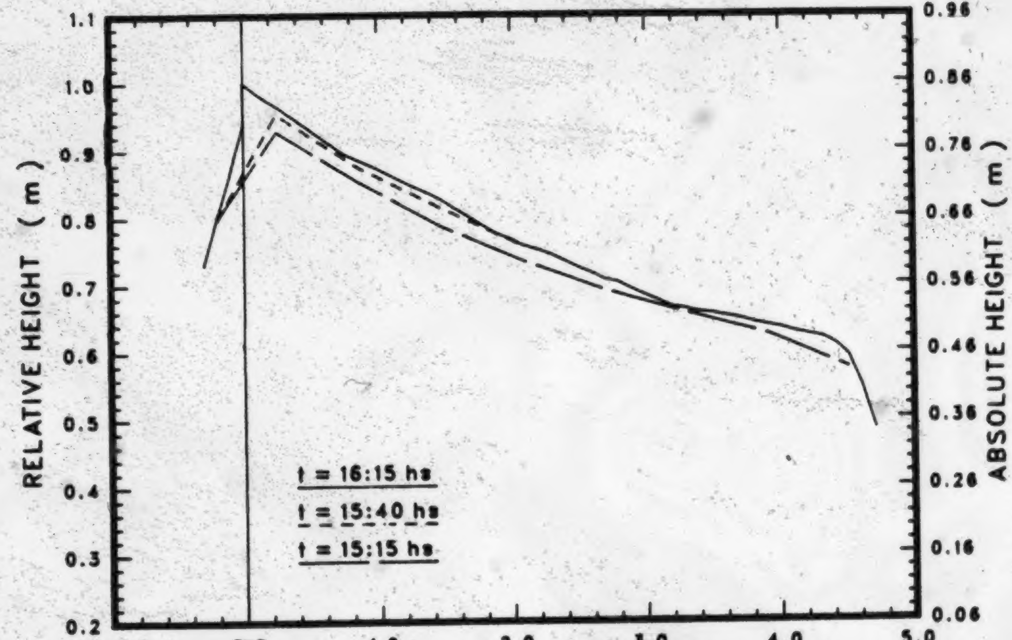


FIG. B.28 – Results of test TS11 (Qt = 5 l/min, Cw = 20 %)

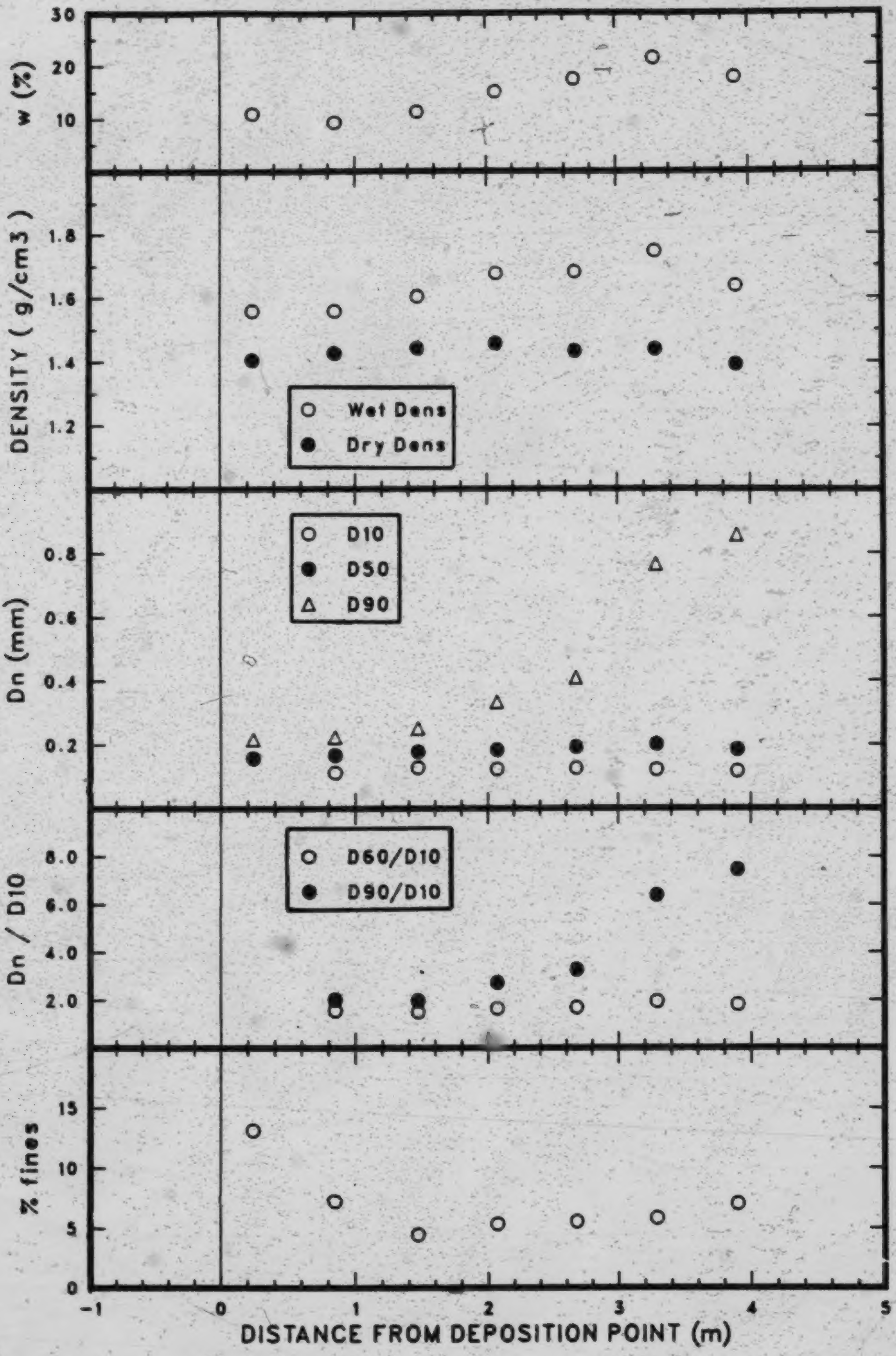


FIG. B.29 – Results of test TS11 (Cont.)

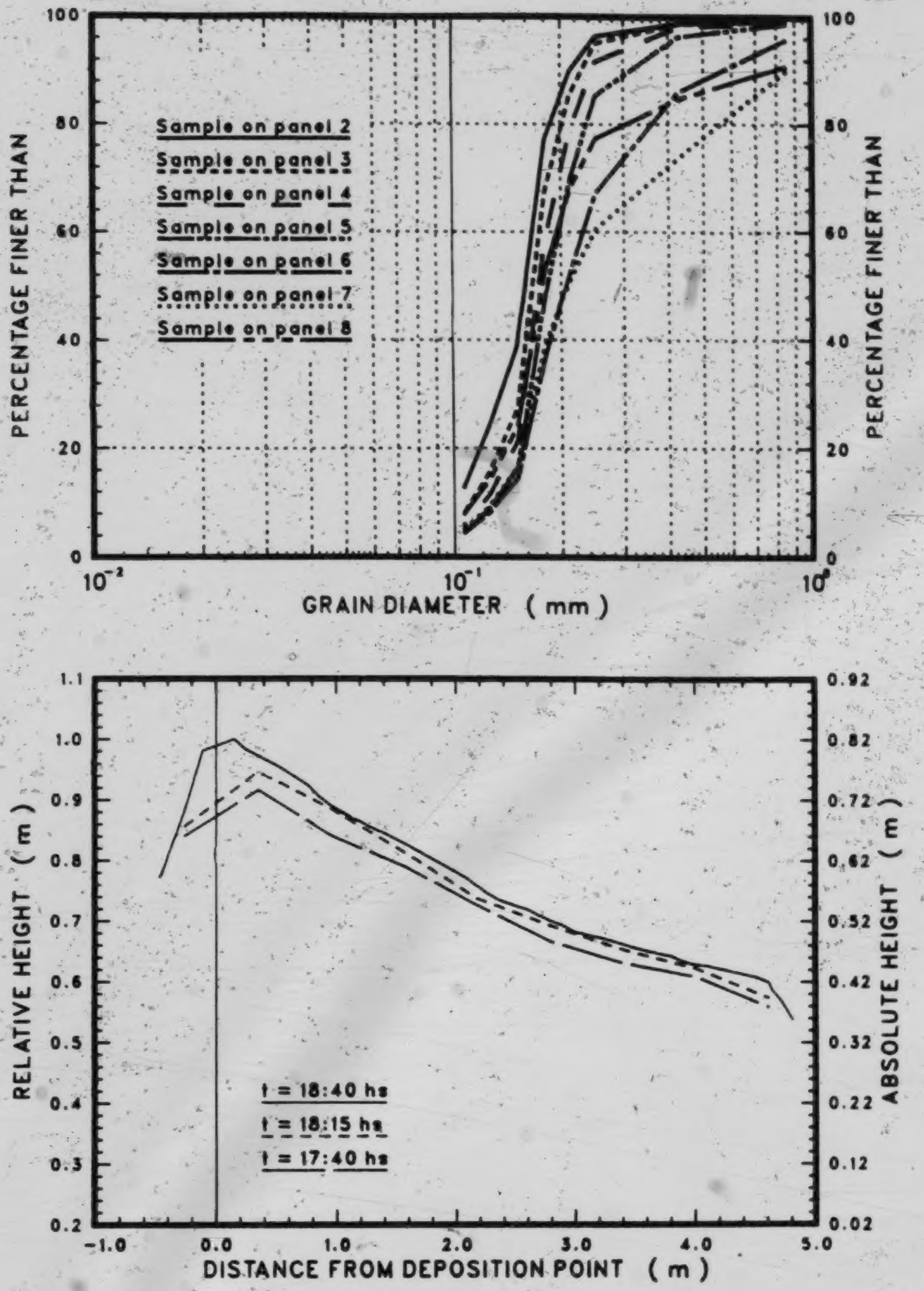


FIG. B.30 – Results of test TS12 (Qt = 5 l/min, Cw = 25 %)

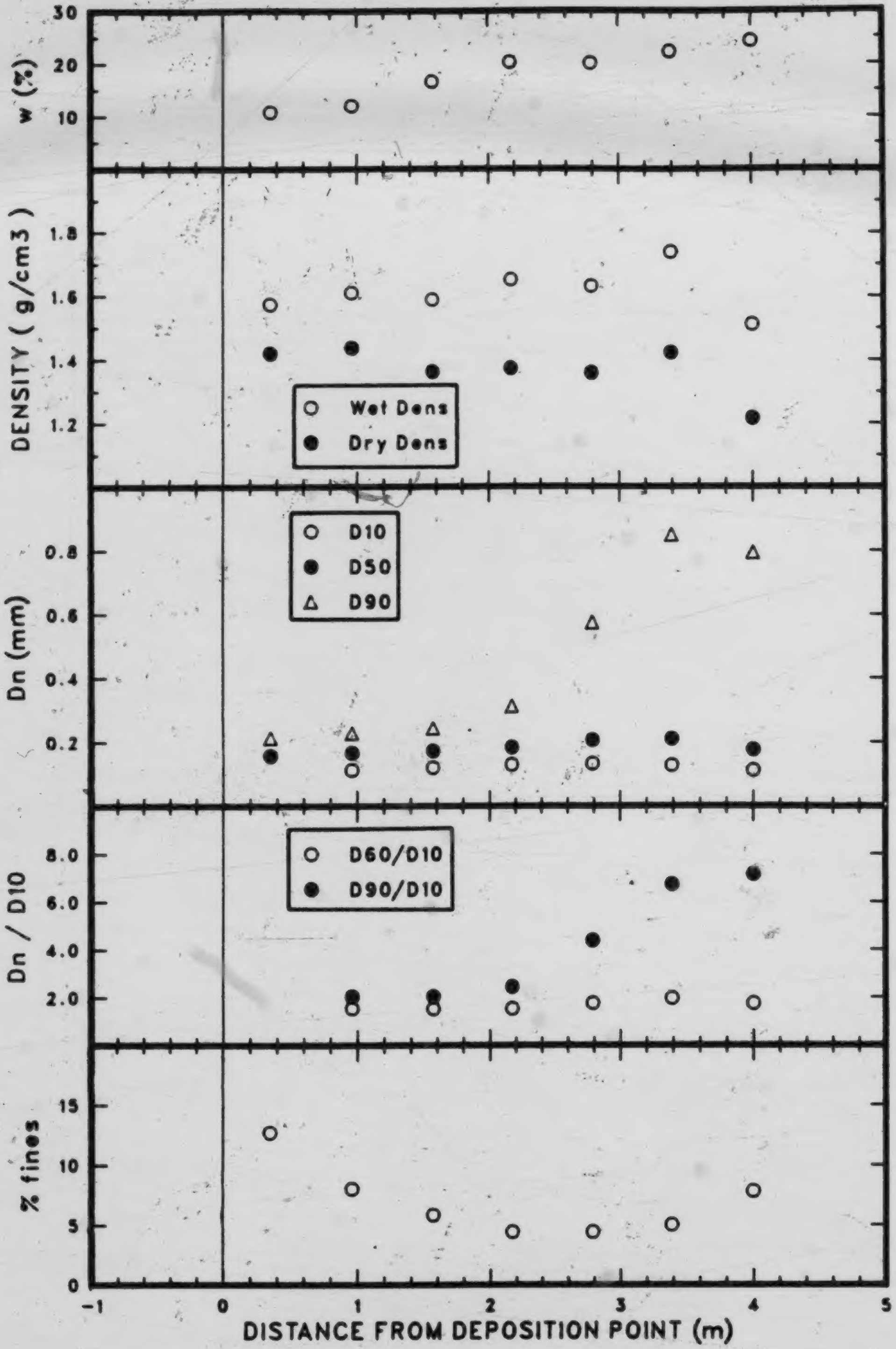


FIG. B.31 – Results of test TS12 (Cont.)

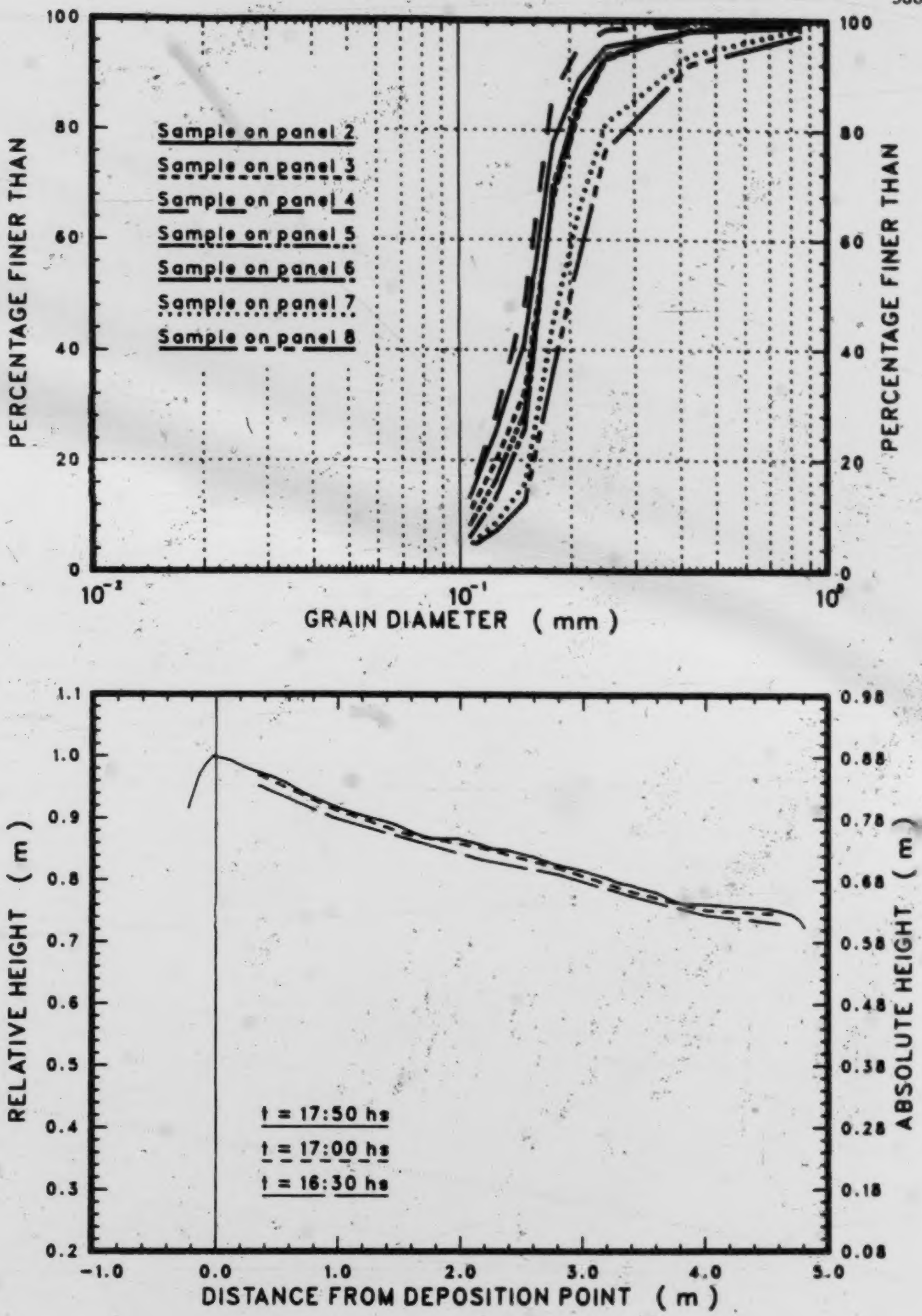


FIG. B.32 – Results of test TS14 (Qt = 10 l/min, Cw = 10 %)

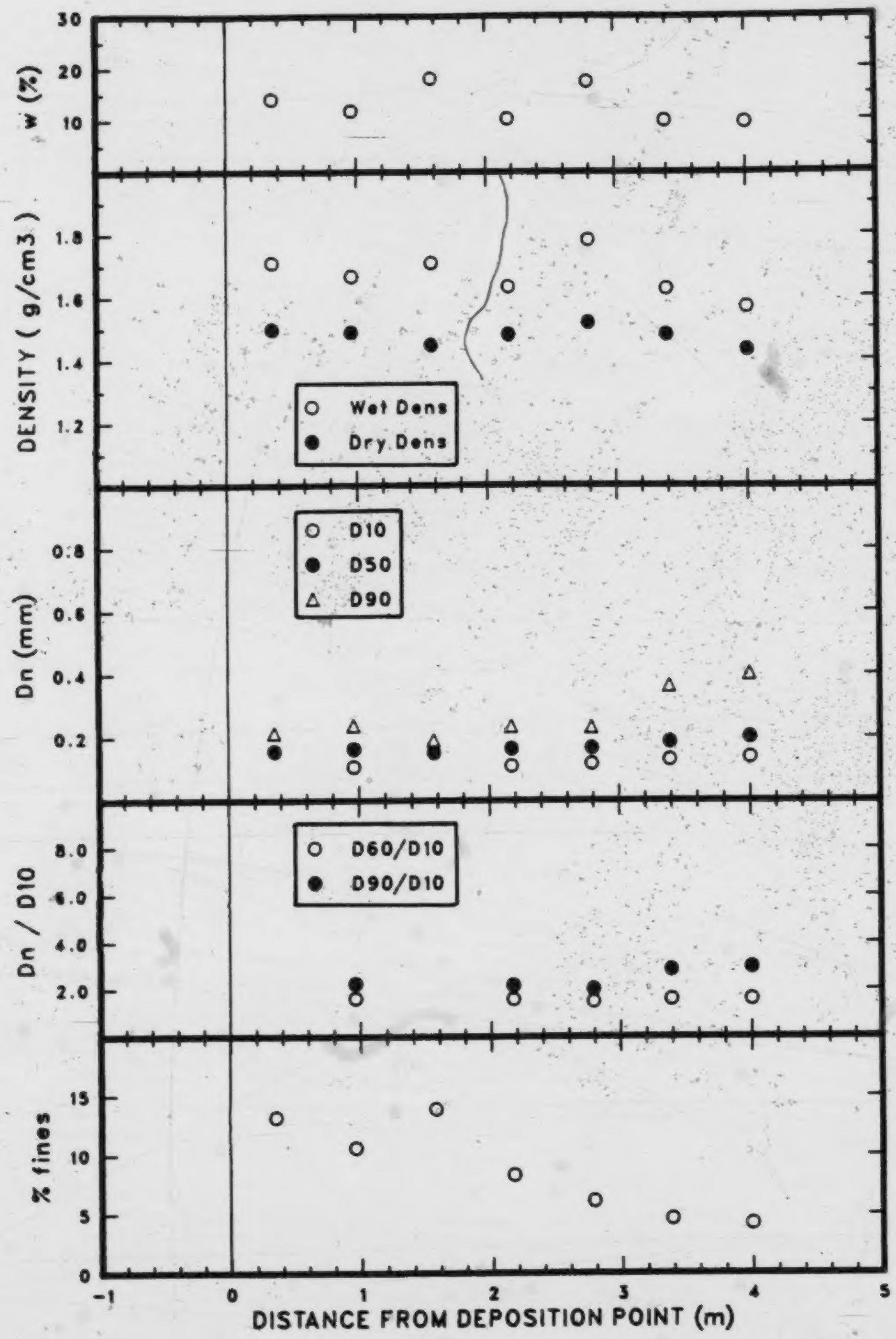


FIG. B.33 – Results of test TS14 (Cont.)

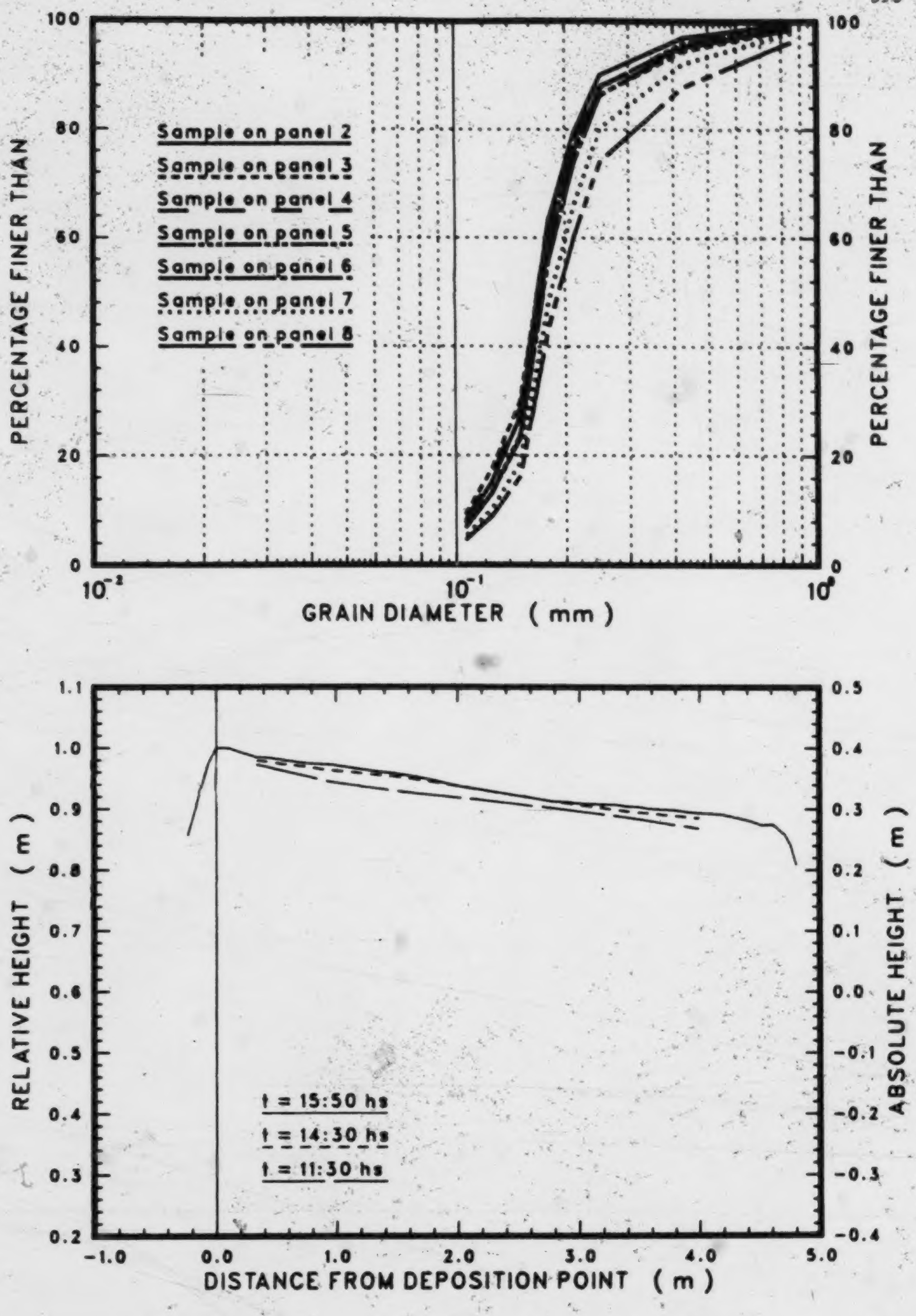


FIG. B.34 – Results of test TS15 (Qt = 10 l/min, Cw = 2 %)

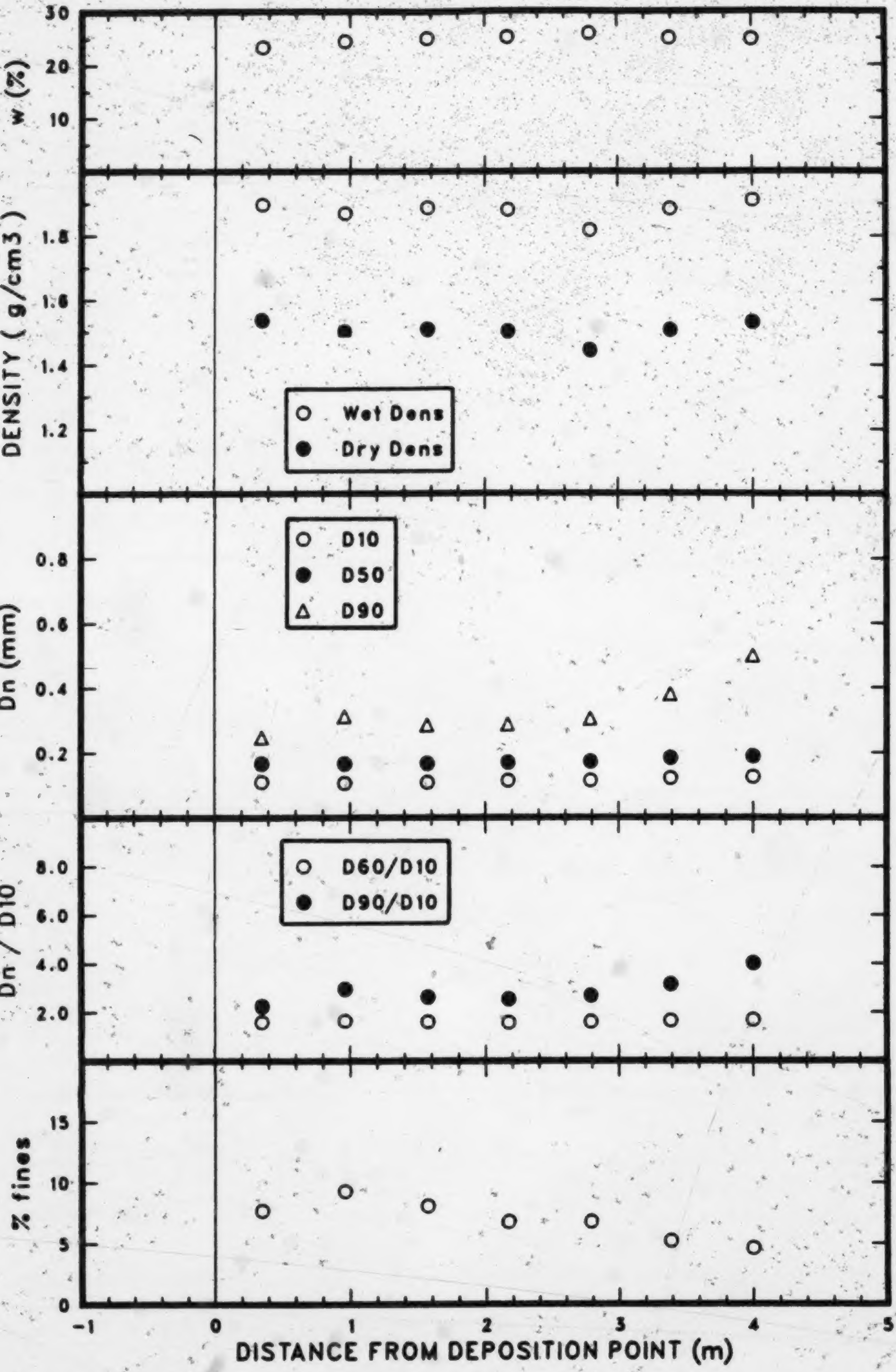
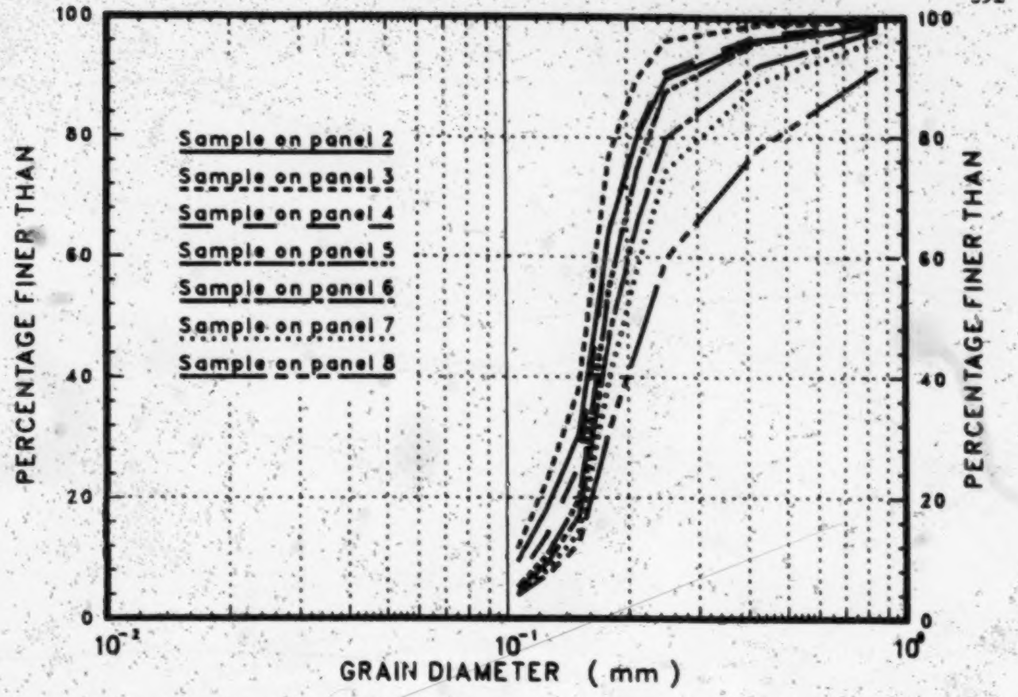


FIG. B.35 – Results of test TS15 (Cont.)



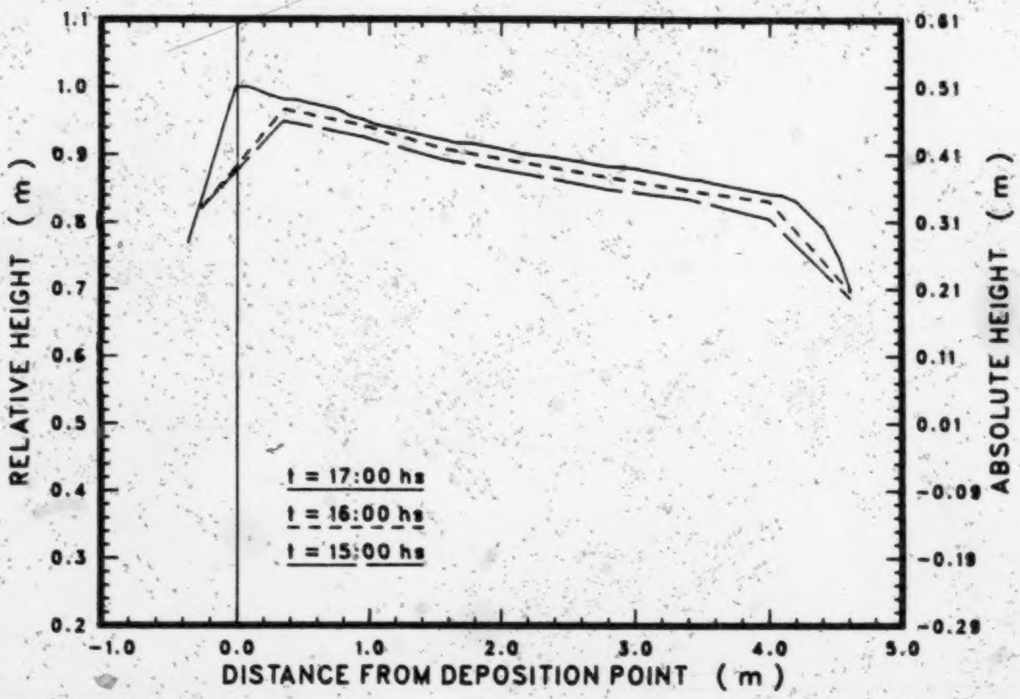


FIG. B.36 – Results of test TS16 (Qt = 10 l/min, Cw = 6 %)

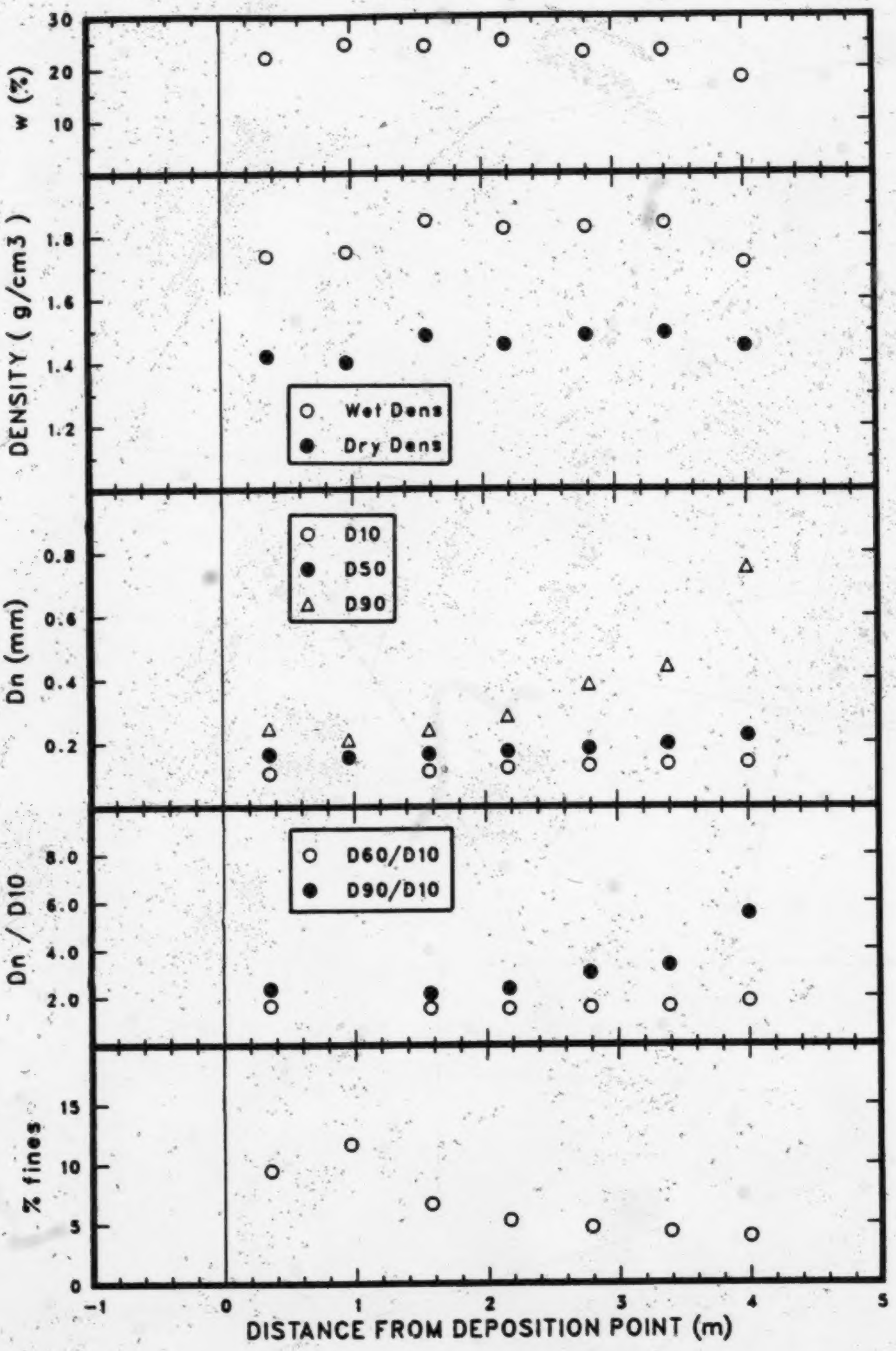
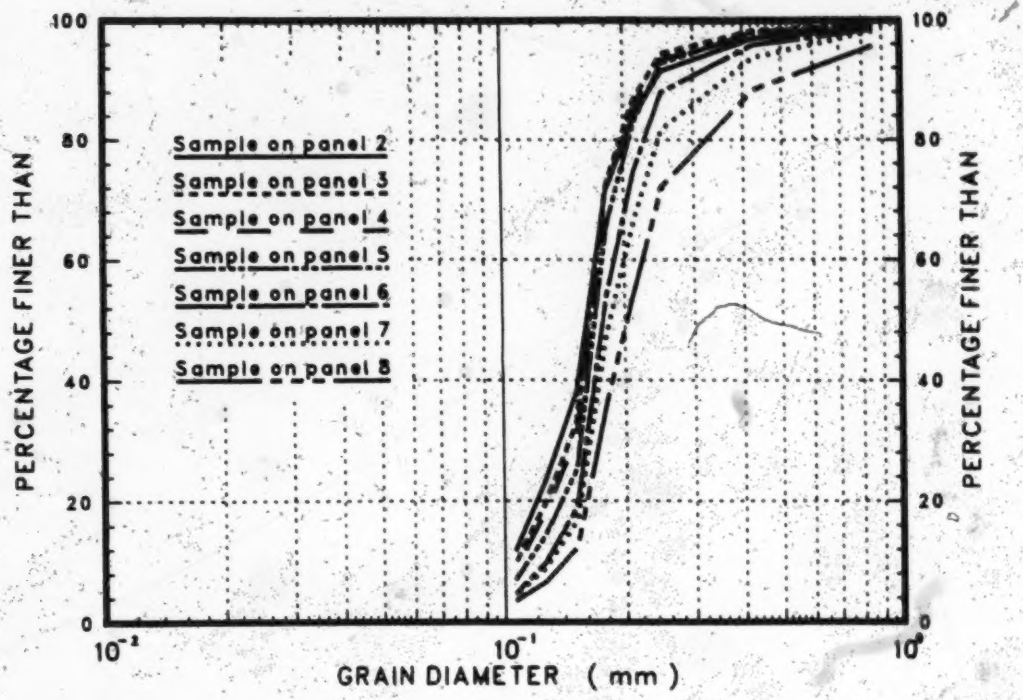


FIG. B.37 – Results of test TS16 (Cont.)



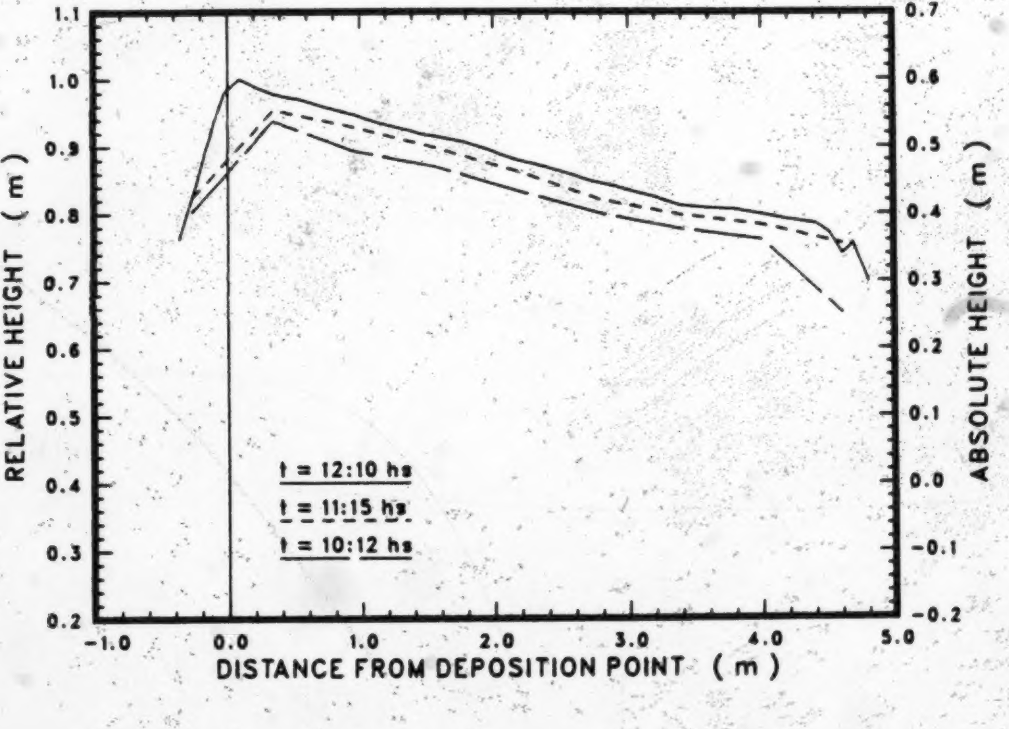


FIG. B.38 – Results of test TS17 (Qt = 10 l/min, Cw = 10 %)

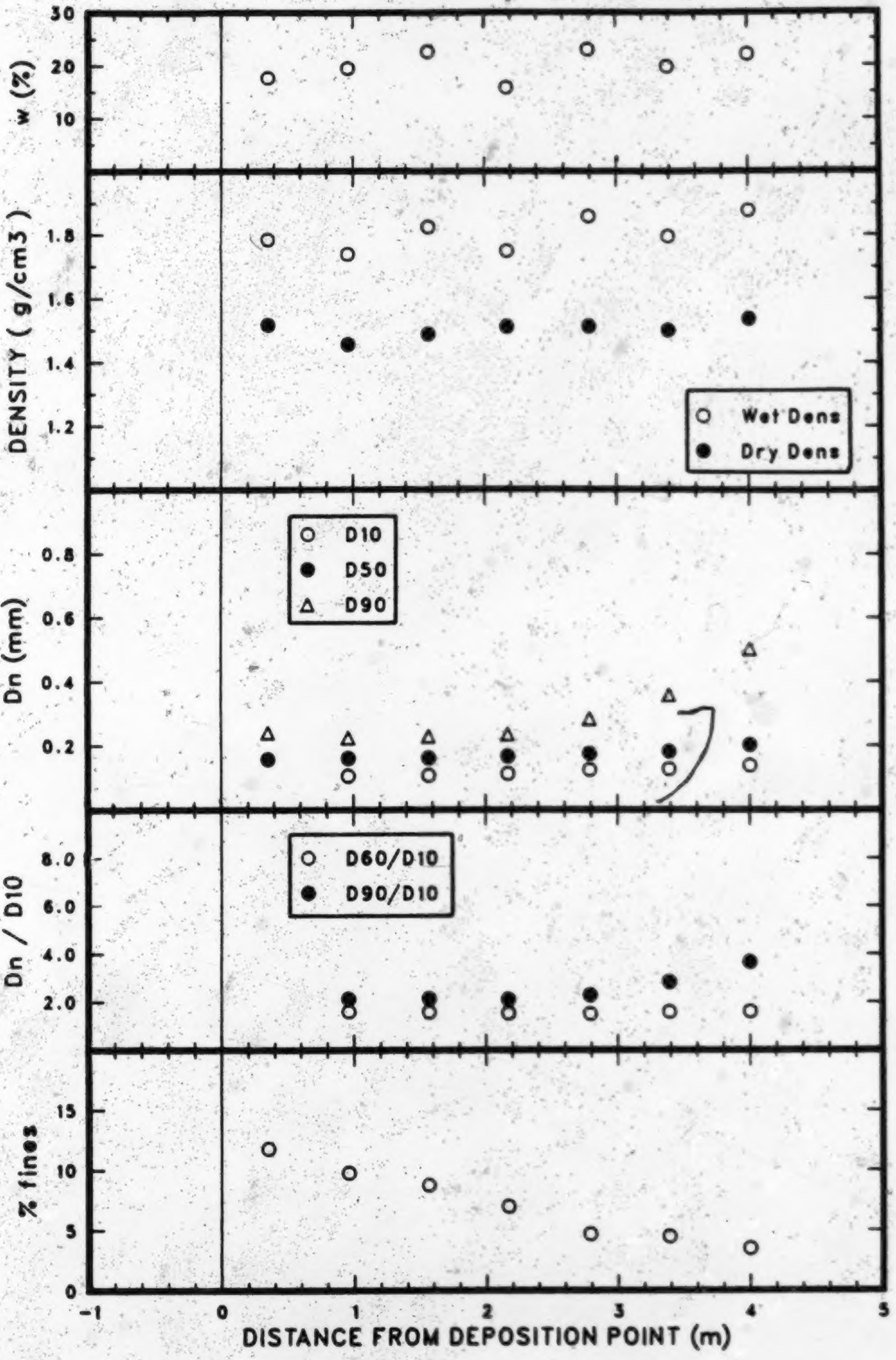


FIG. B.39 – Results of test TS17 (Cont.)

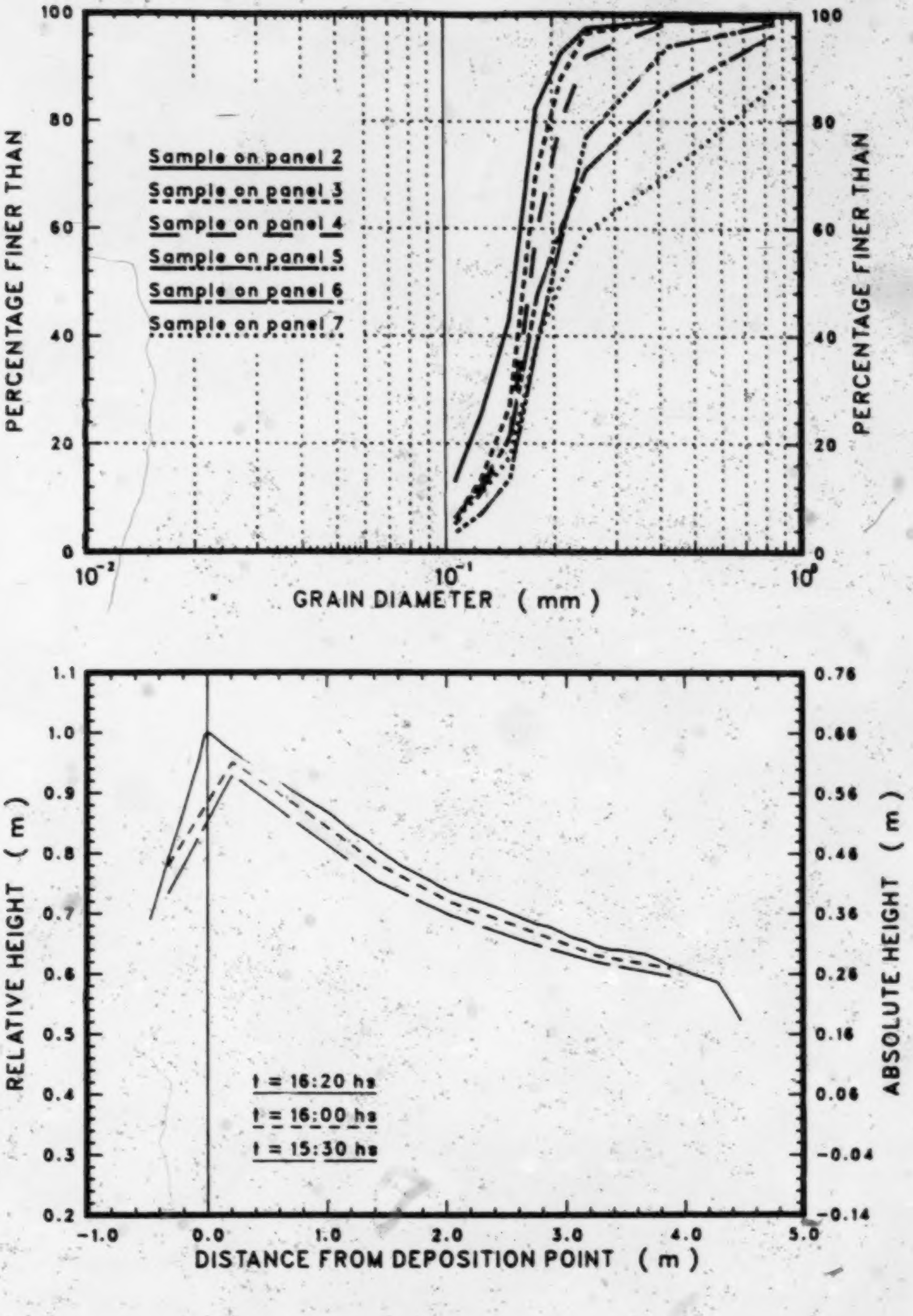


FIG. B.40 – Results of test TS18 (Qt = 5 l/min, Cw = 30 %)

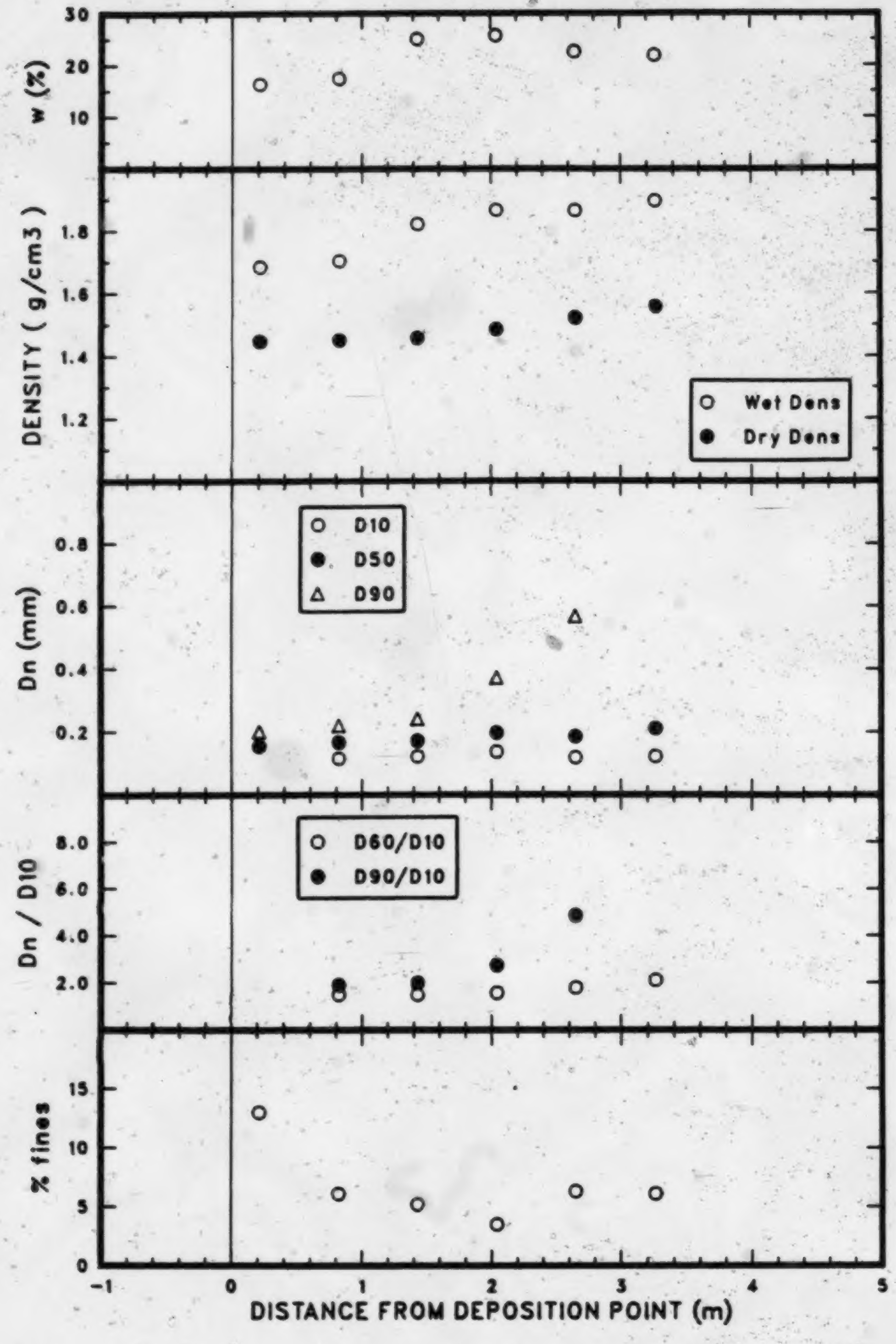
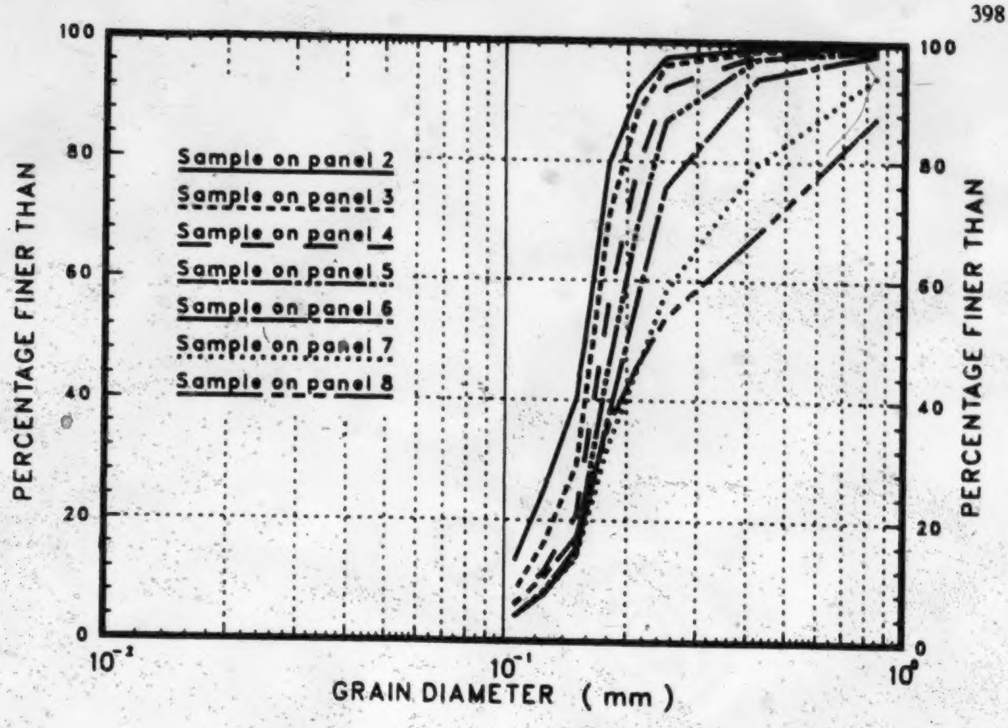


FIG. B.41 – Results of test TS18 (Cont.)



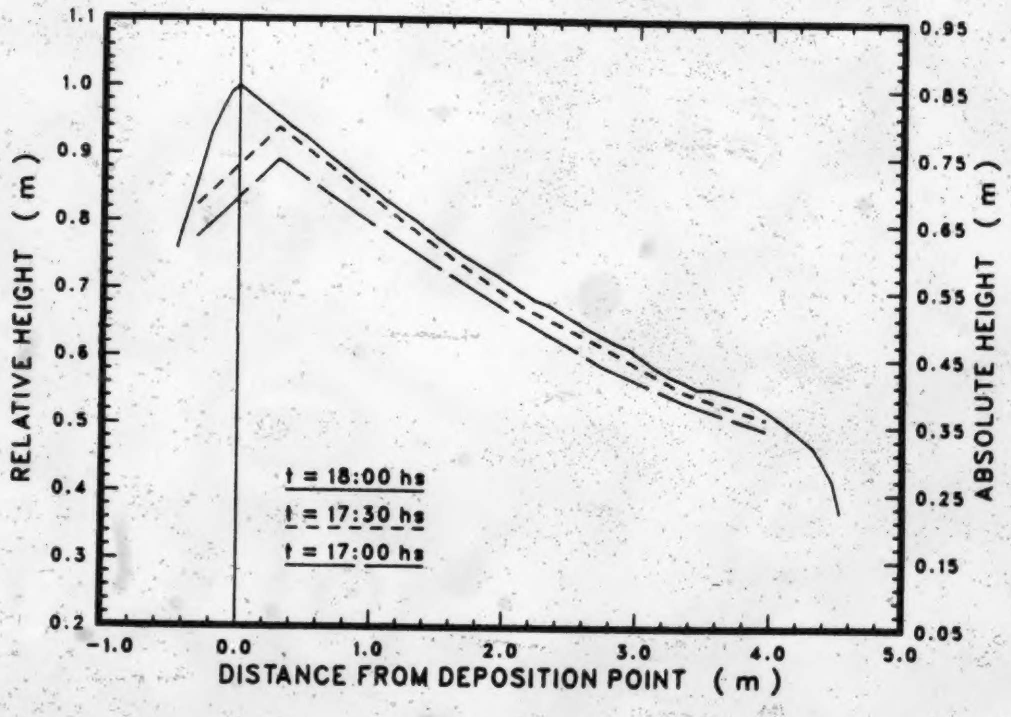


FIG. B.42 – Results of test TS19 (Qt = 5 l/min, Cw = 35 %)

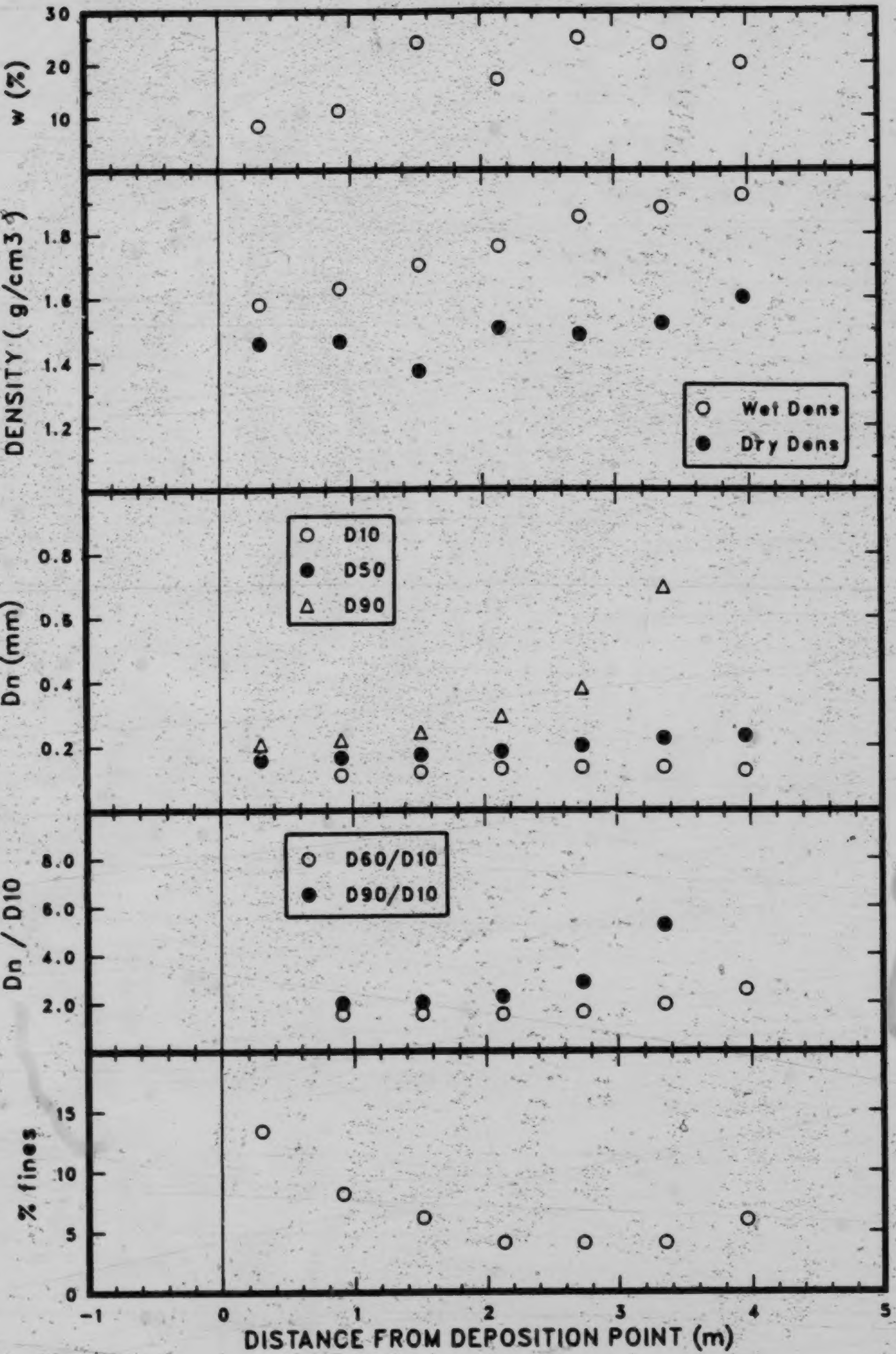


FIG. B.43 – Results of test TS19 (Cont.)

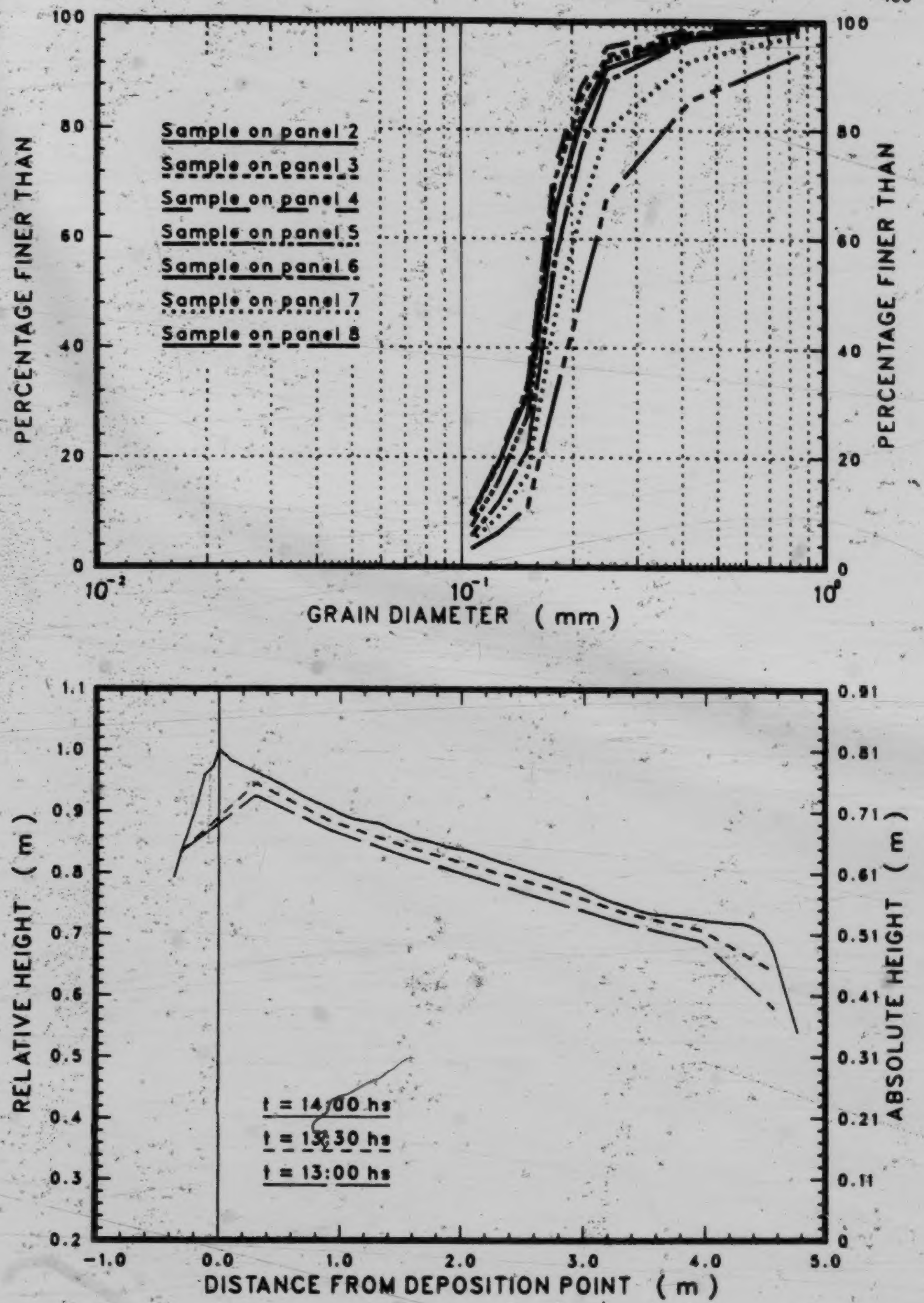


FIG. B.44 – Results of test TS20 (Qt = 10 l/min, Cw = 15 %)

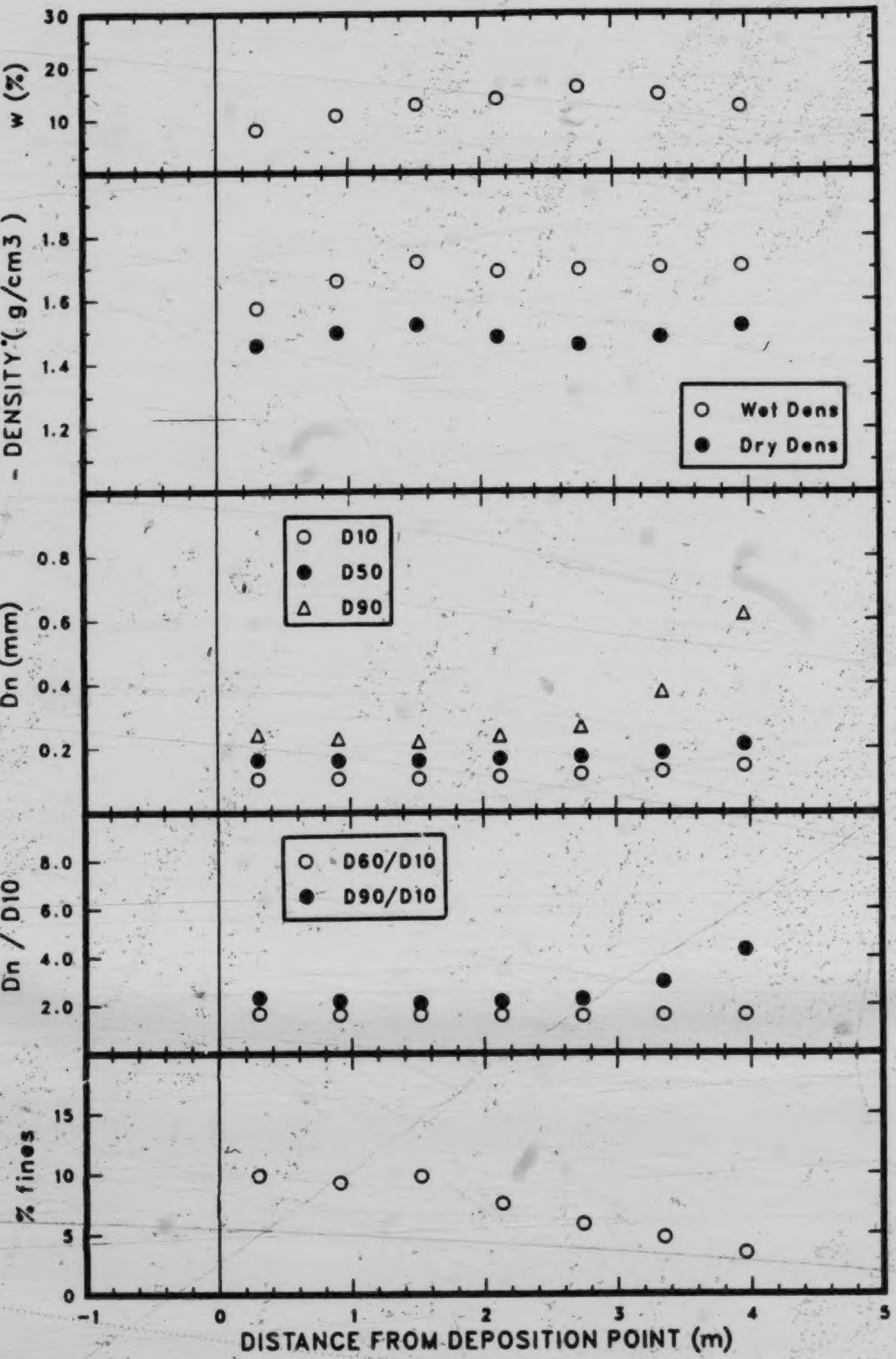


FIG. B.45 – Results of test TS20 (Cont.)

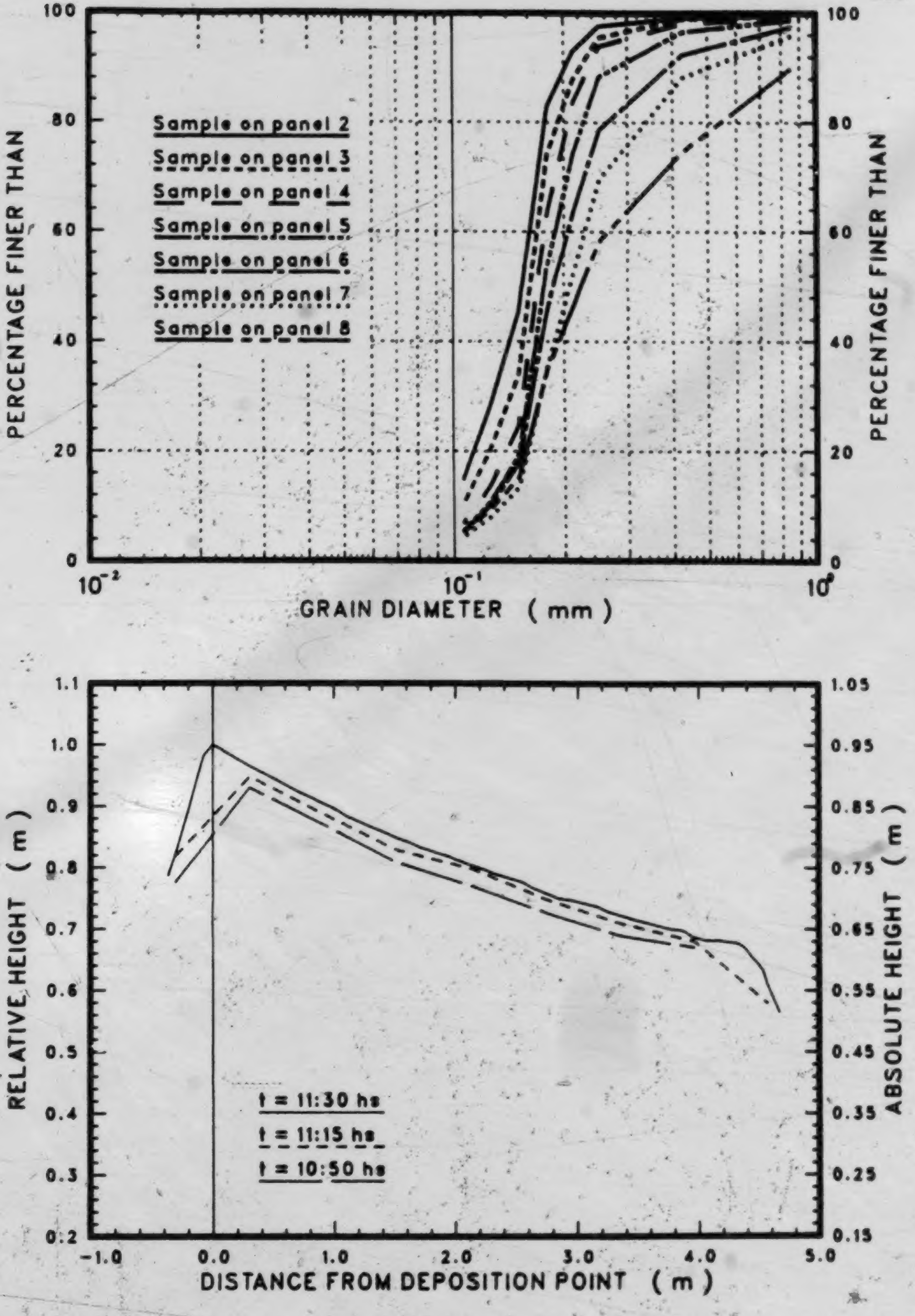


FIG. B.46 – Results of test TS21 (Qt = 10 l/min, Cw = 20 %)

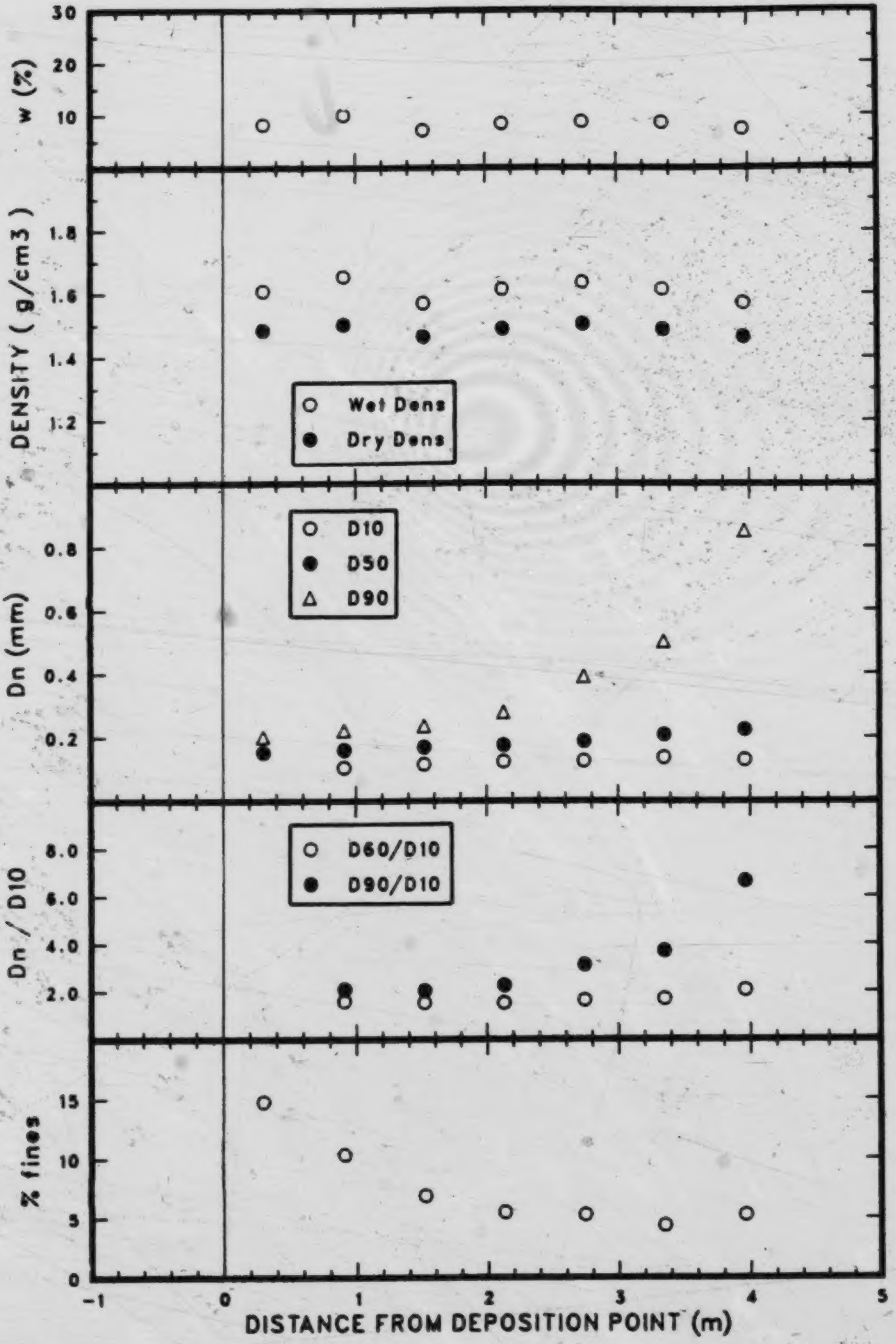
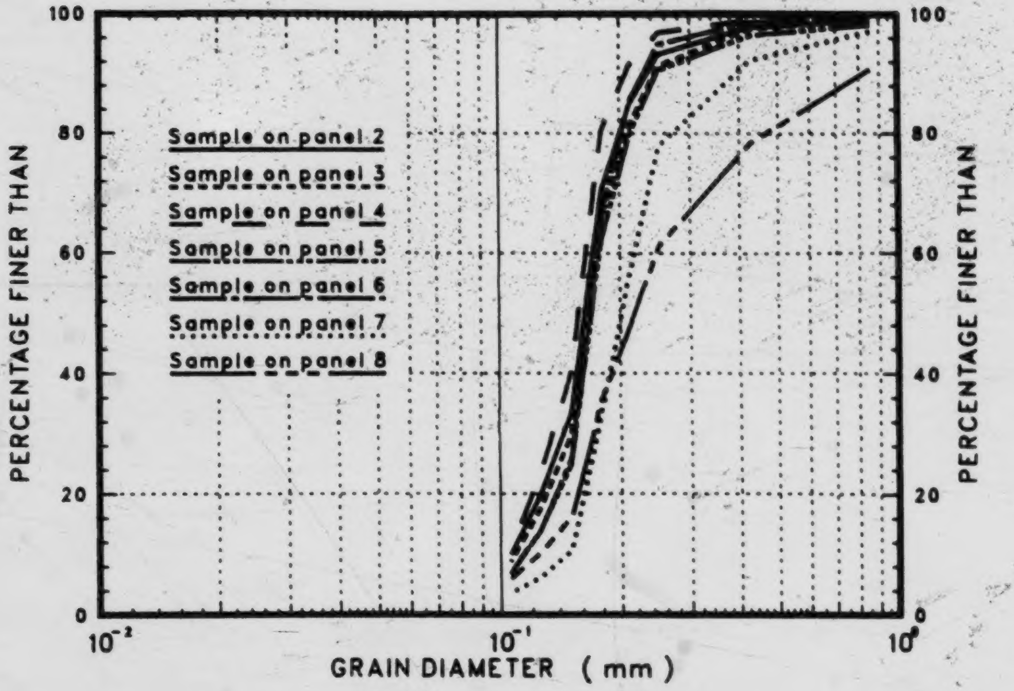


FIG. B.47 – Results of test TS21 (Cont.)



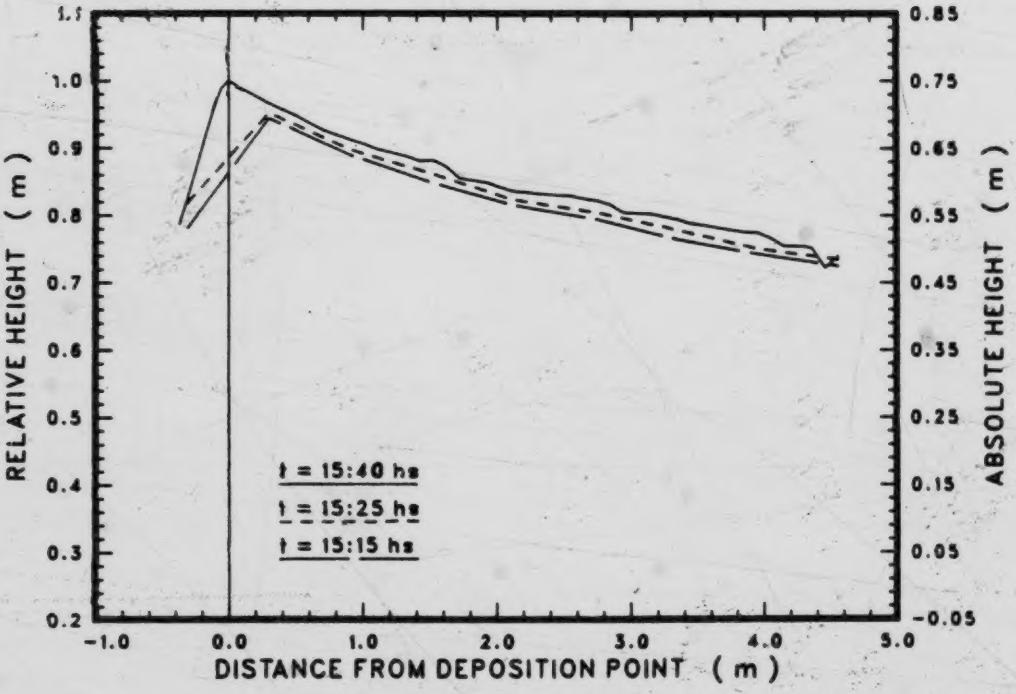


FIG. B.48 – Results of test TS22 (Qt = 15 l/min, Cw = 15 %)

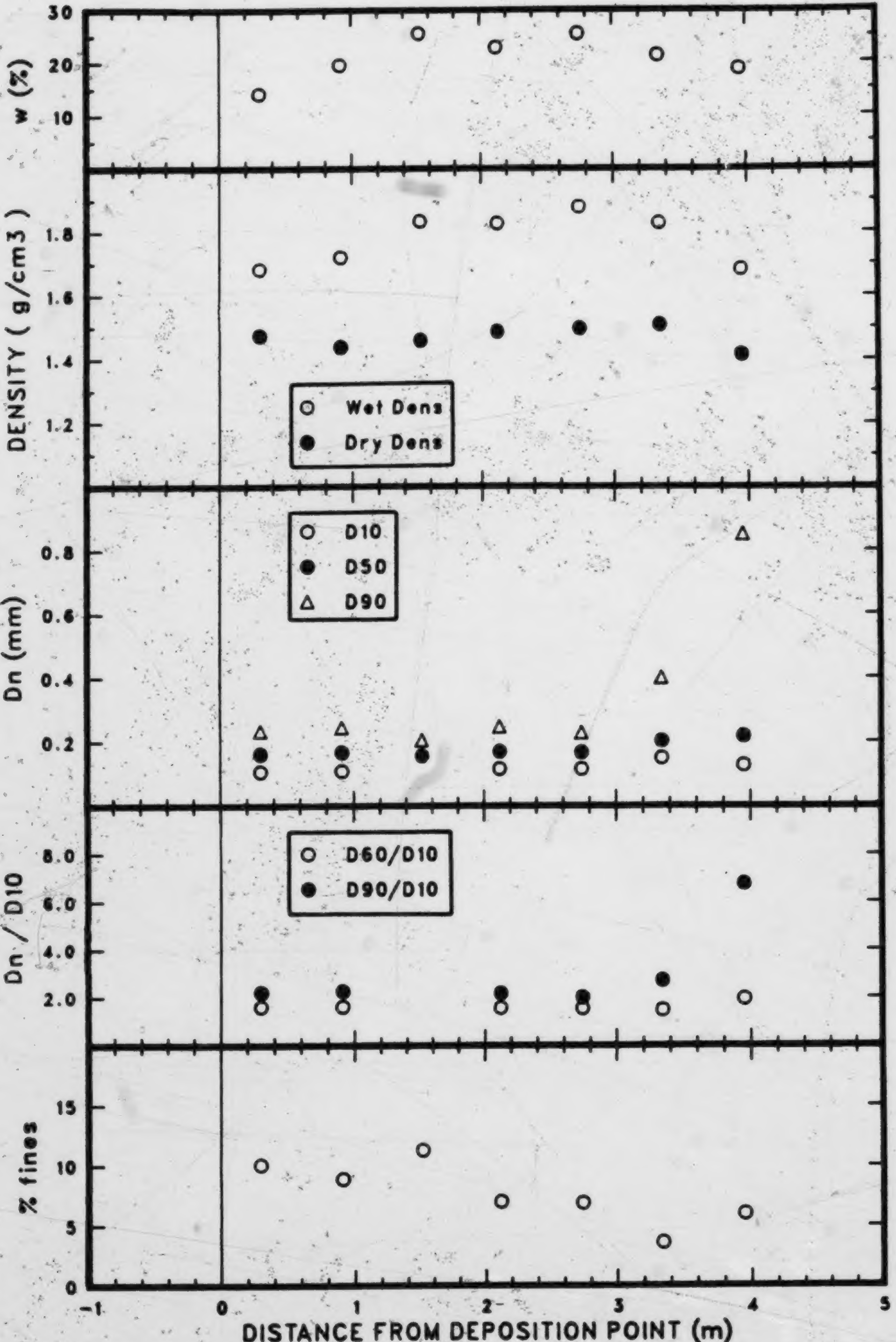


FIG. B.49 – Results of test TS22 (Cont.)

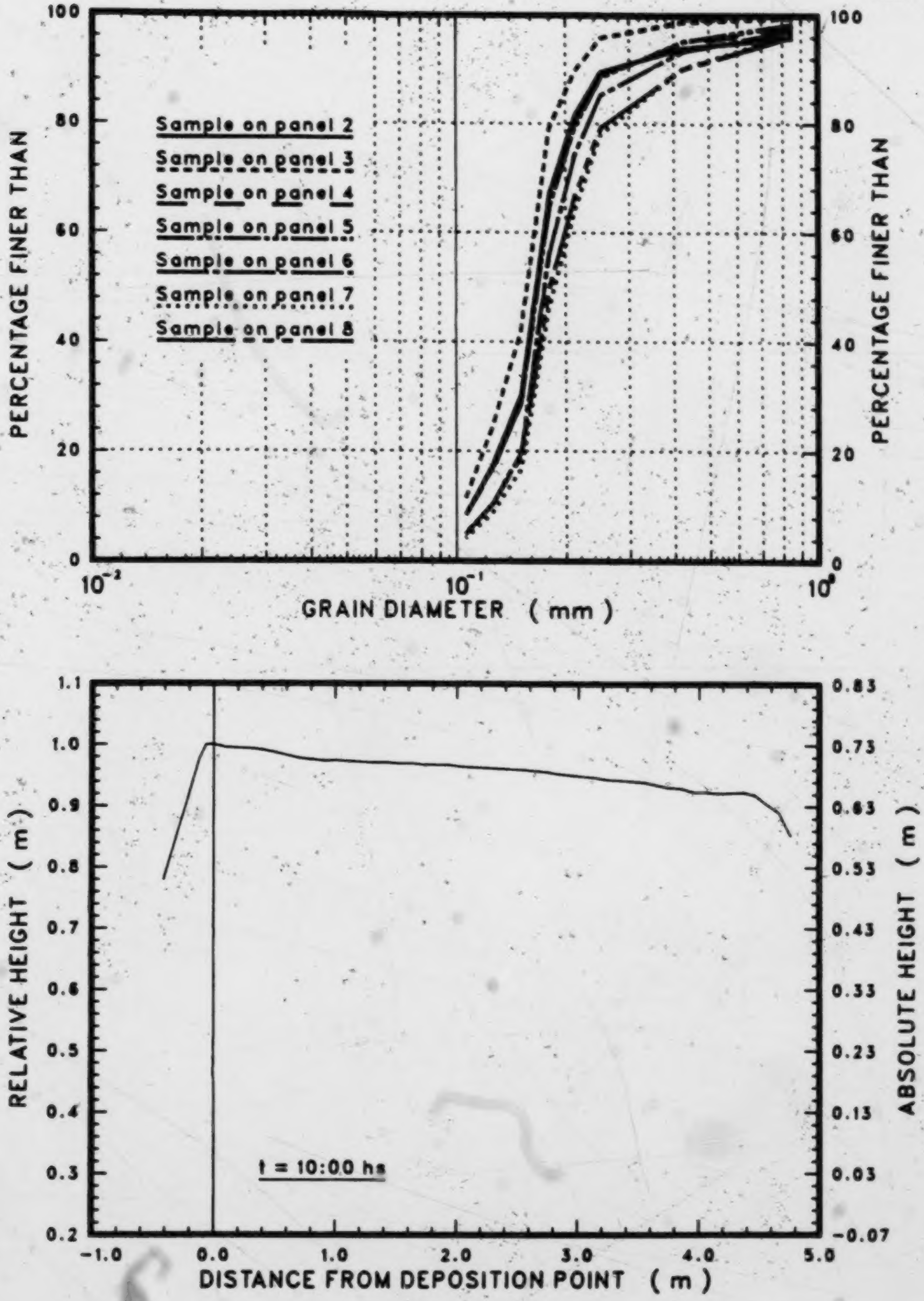


FIG. B.50 – Results of test TS23 (Qt = 20 l/min, Cw = 2 %)

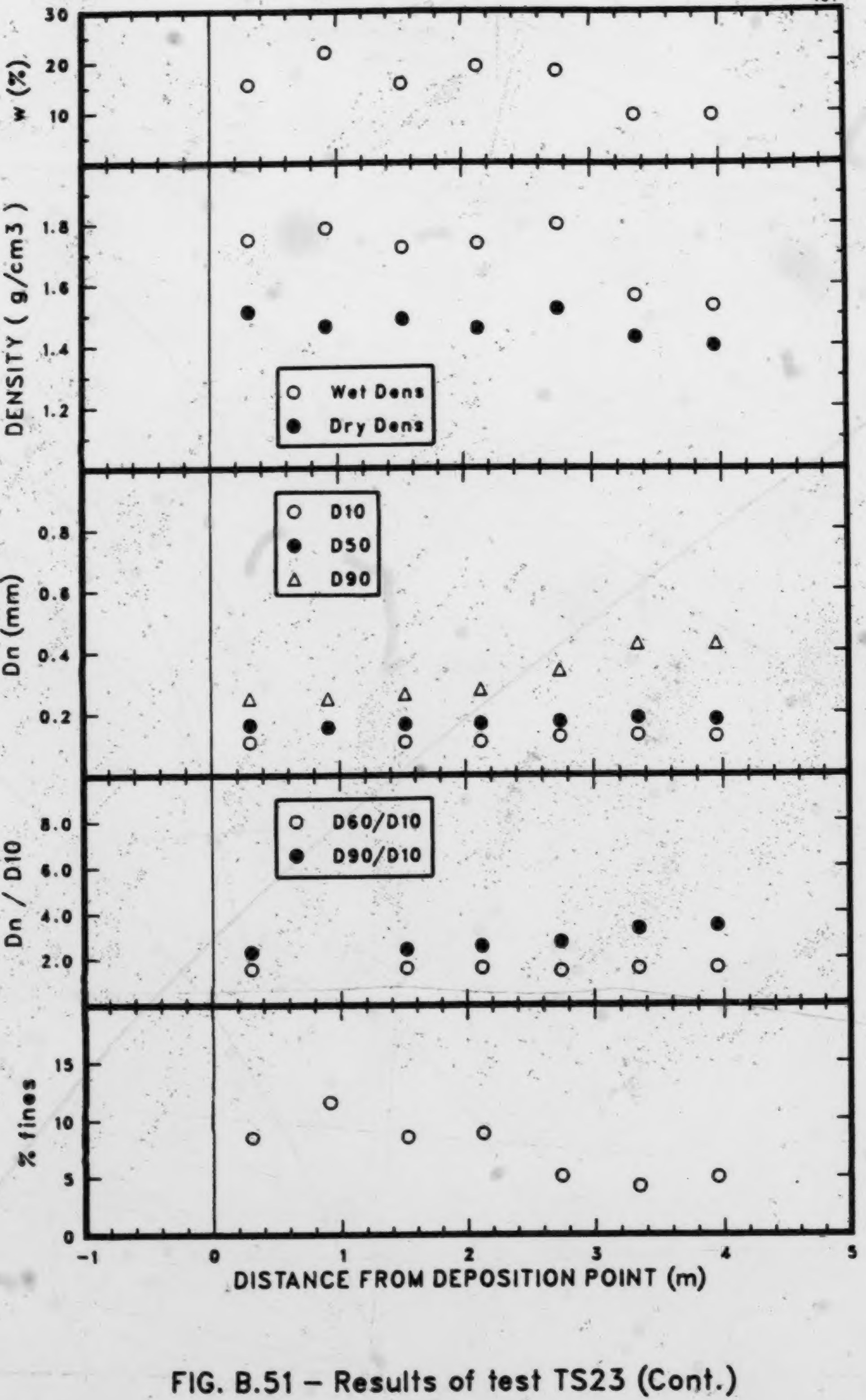


FIG. B.51 – Results of test TS23 (Cont.)

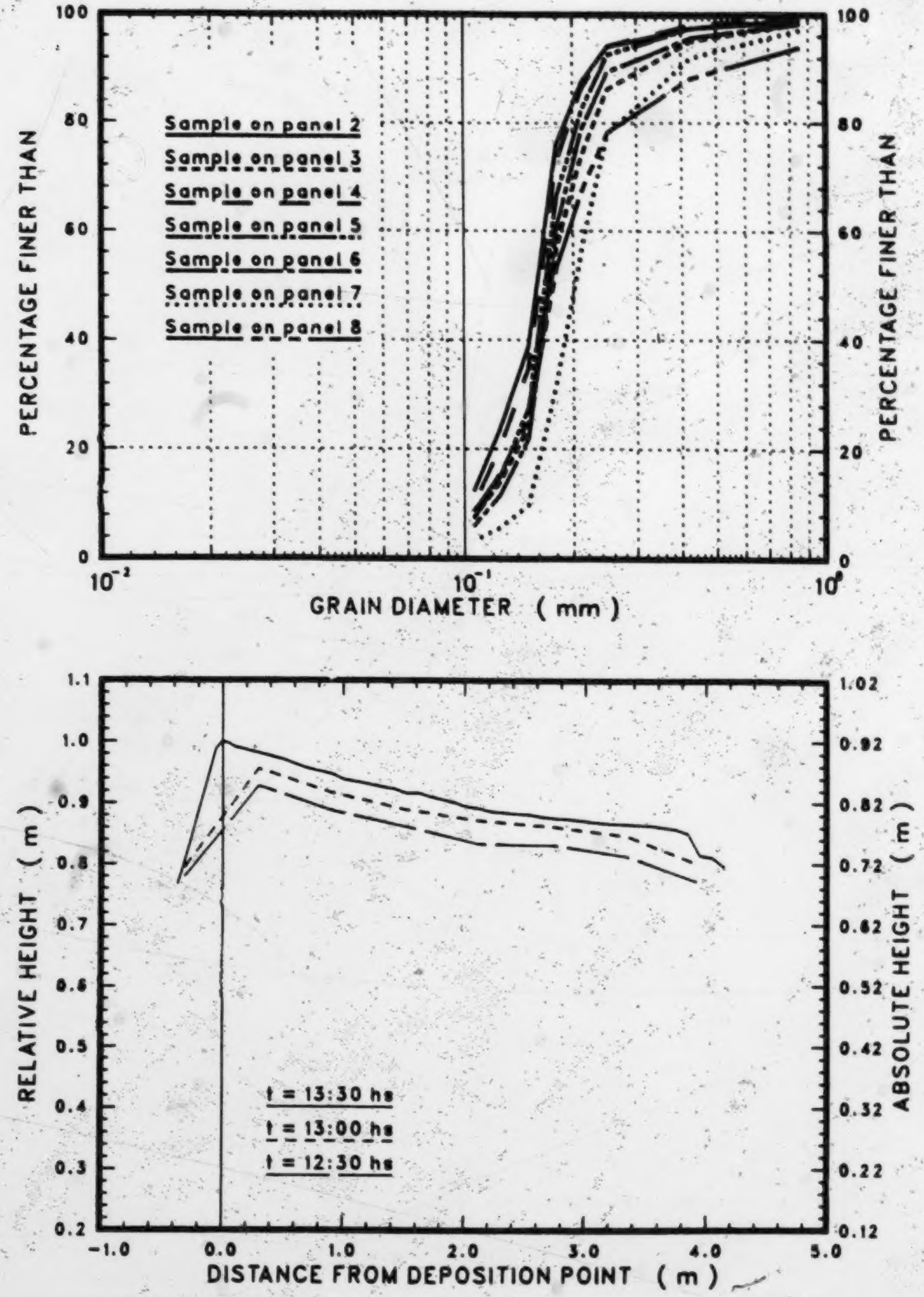


FIG. B.52 – Results of test TS24 (Qt = 20 l/min, Cw = 10 %)

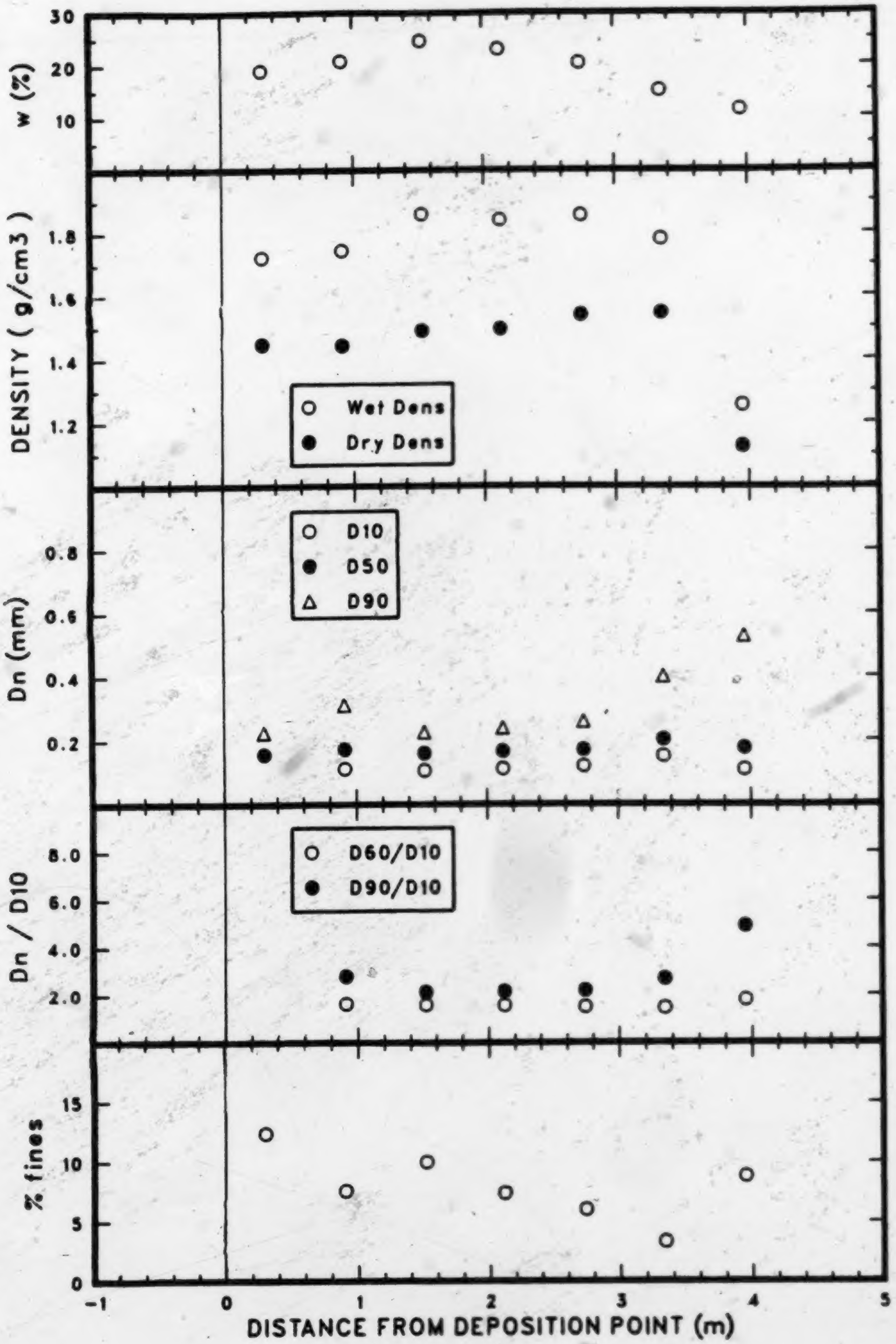


FIG. B.53 – Results of test TS24 (Cont.)

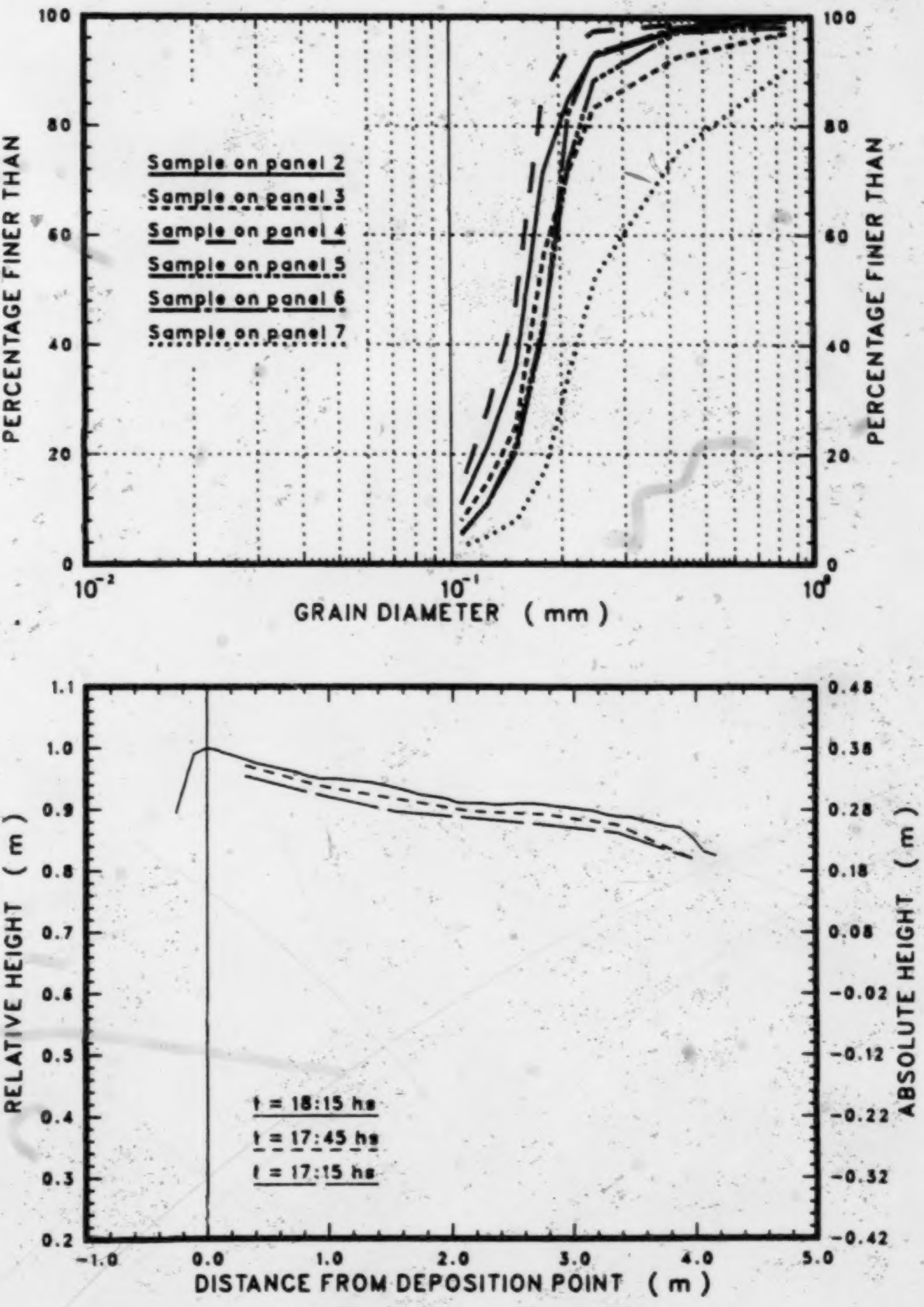


FIG. B.54 – Results of test TS25 (Qt = 20 l/min, Cw = 6 %)

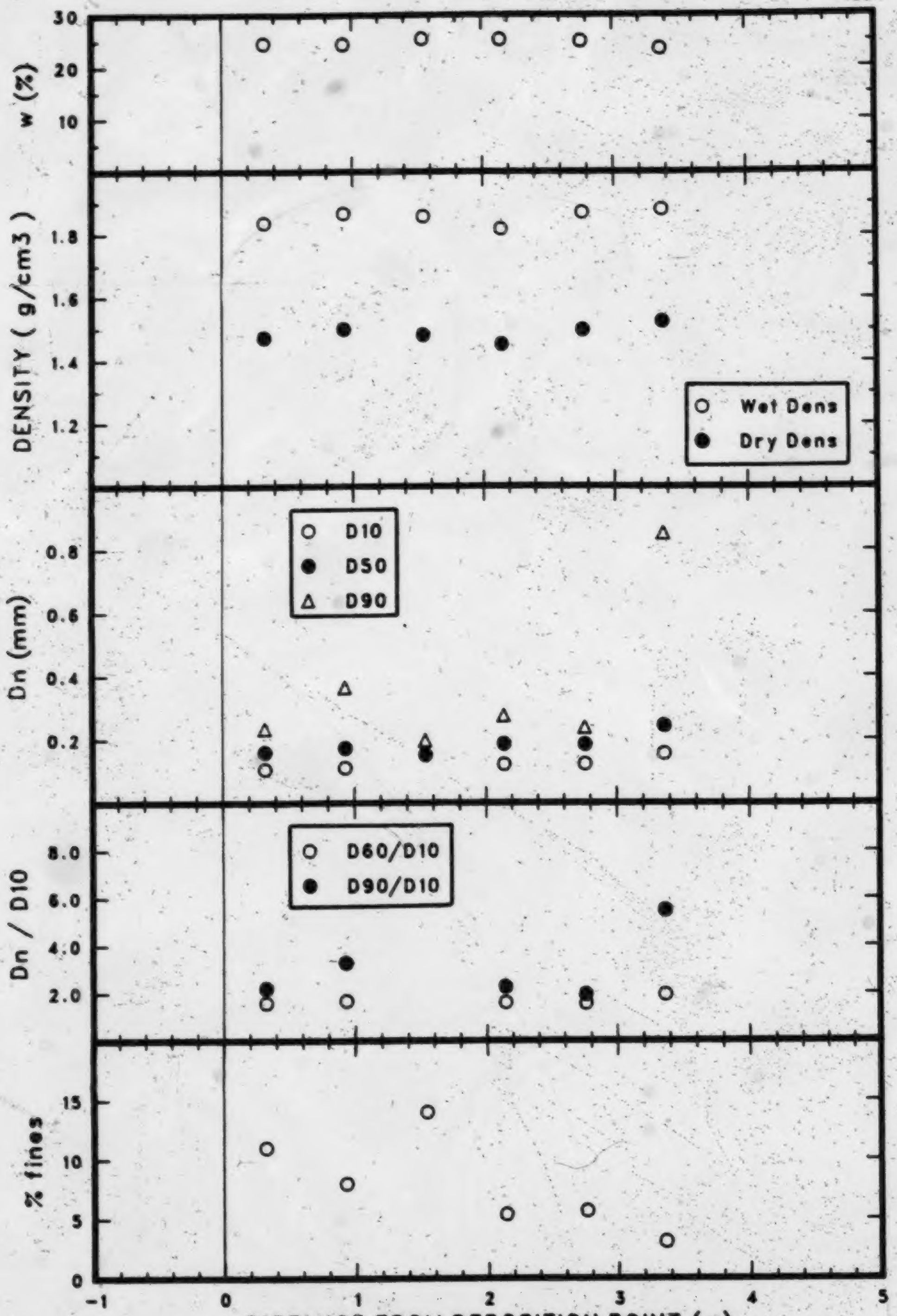


FIG. B.55 – Results of test TS25 (Cont.)

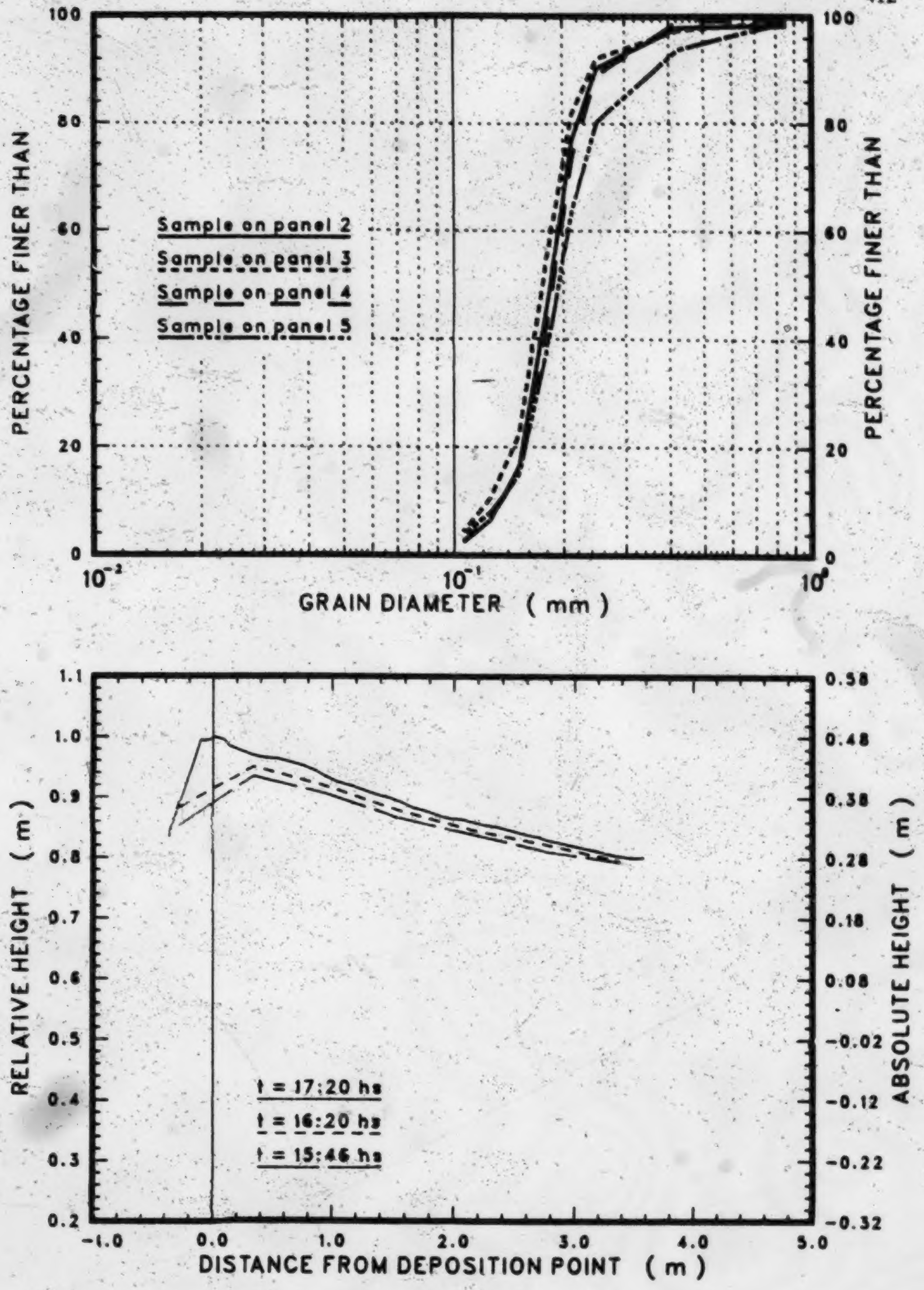


FIG. B.56 – Results of test TS26 (Qt = 5 l/min, Cw = 10 %)

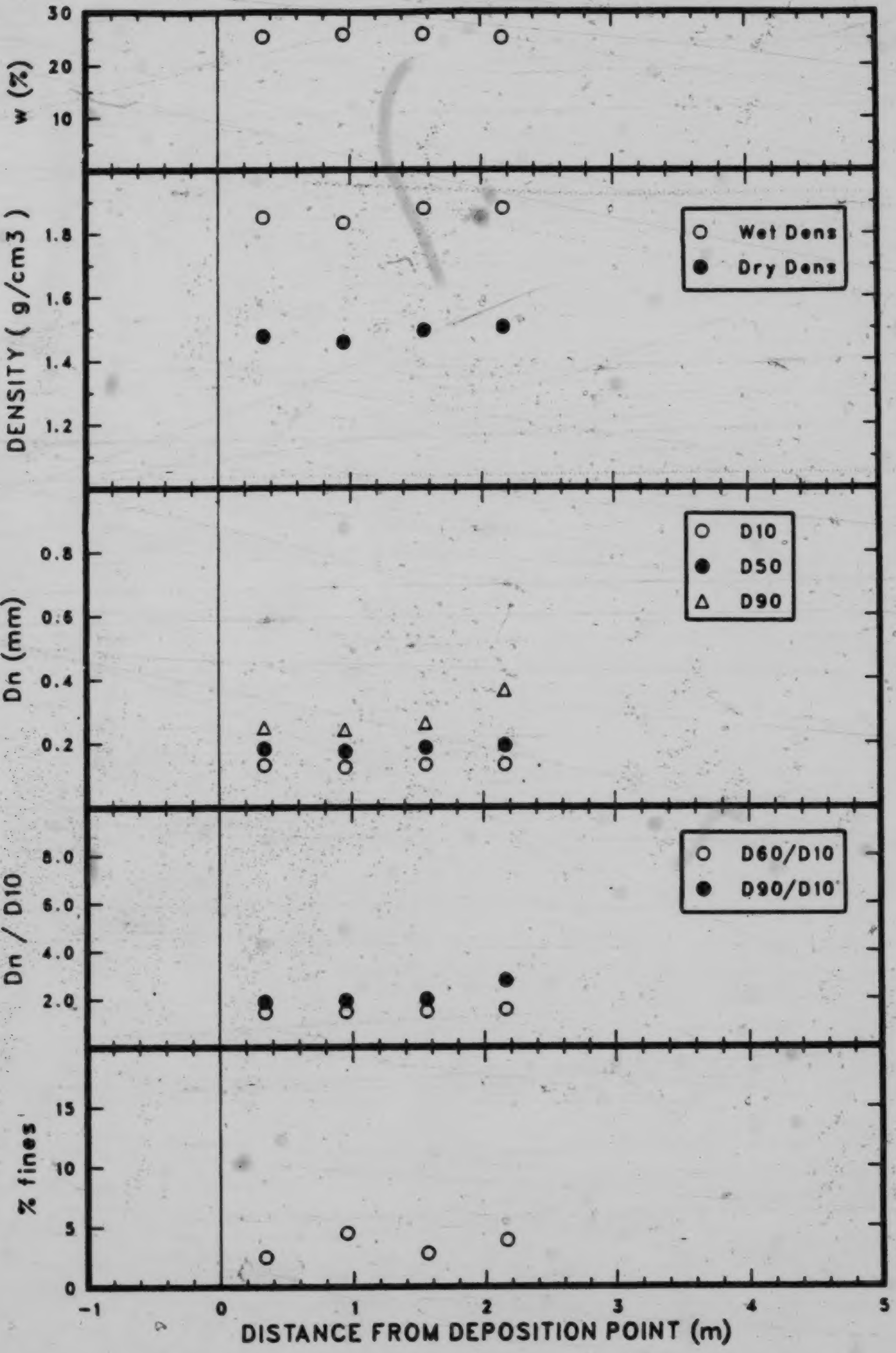


FIG. B.57 – Results of test TS26 (Cont.)

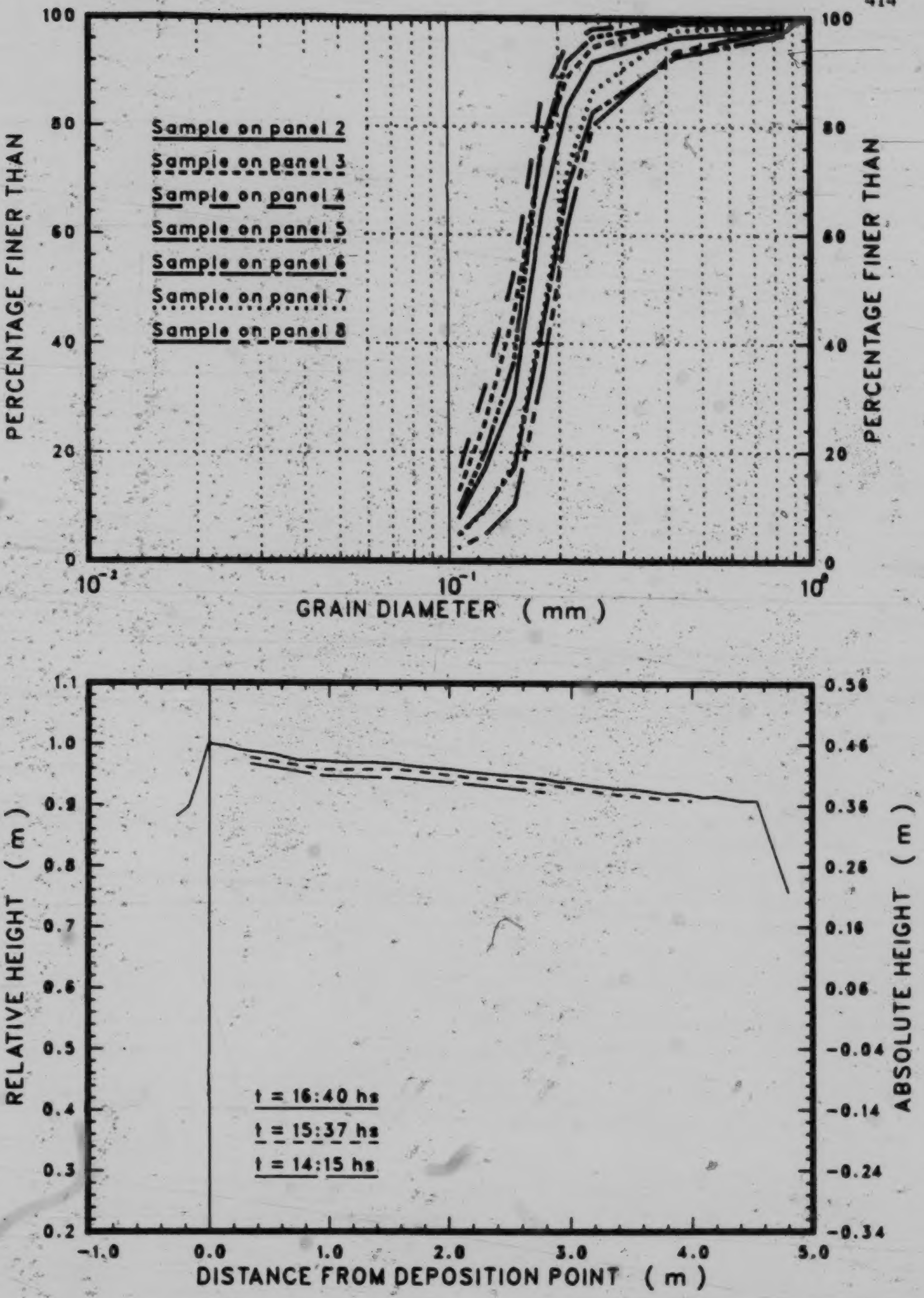


FIG. B.58 – Results of test TS27 (Qt = 20 l/min, Cw = 2 %)

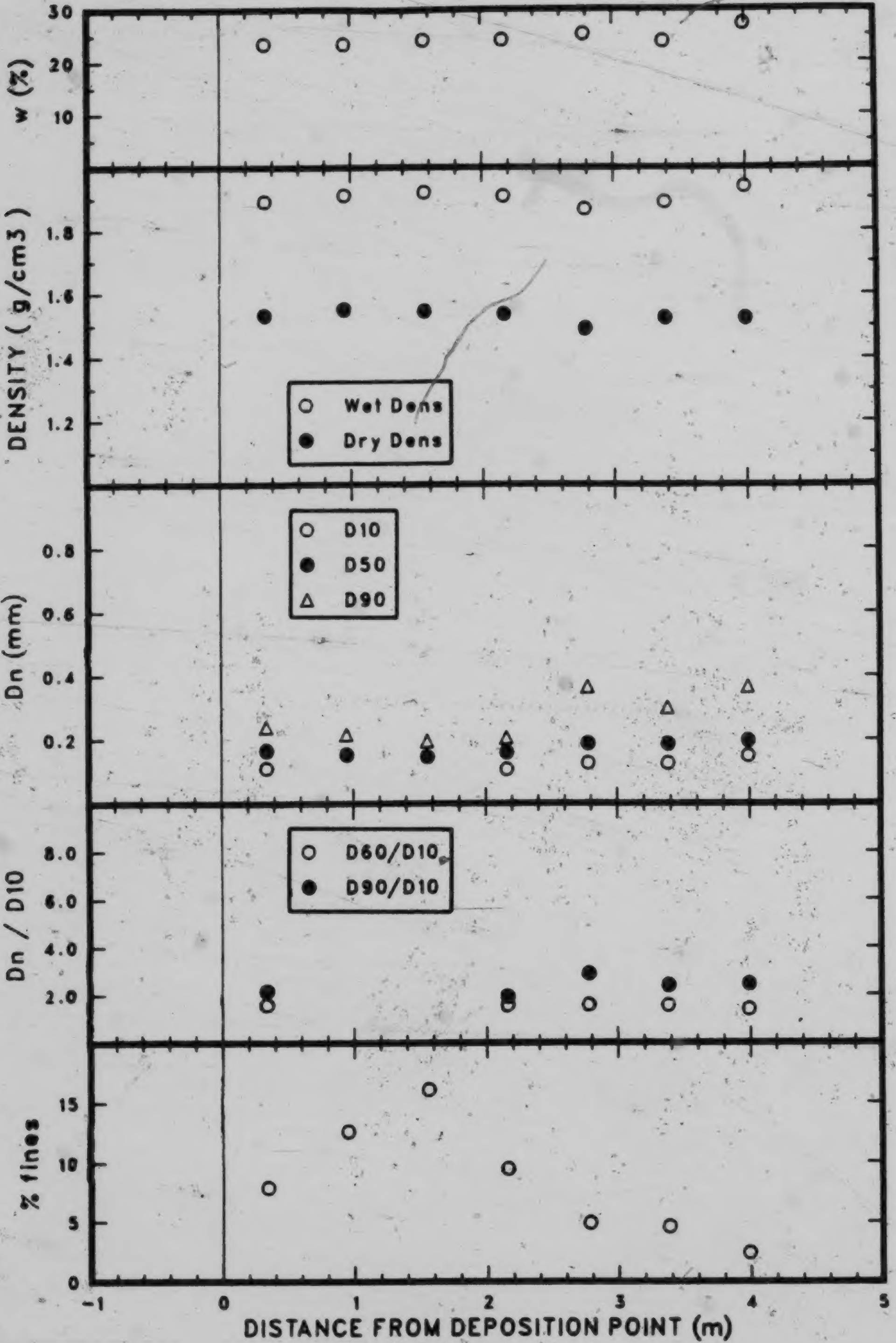


FIG. B.59 – Results of test TS27 (Cont.)

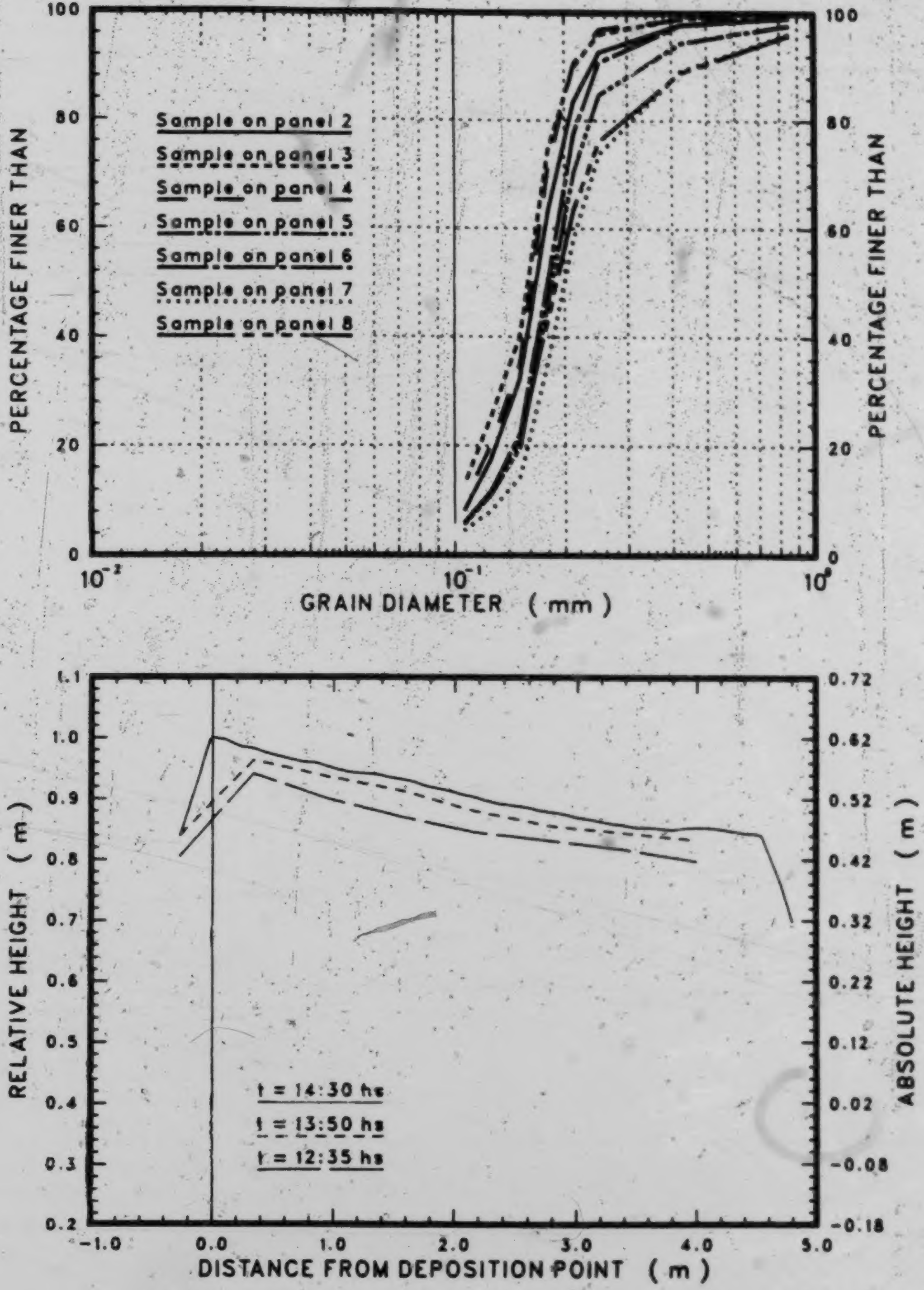


FIG. B.60 – Results of test TS28 (Qt = 15 l/min, Cw = 10 %)

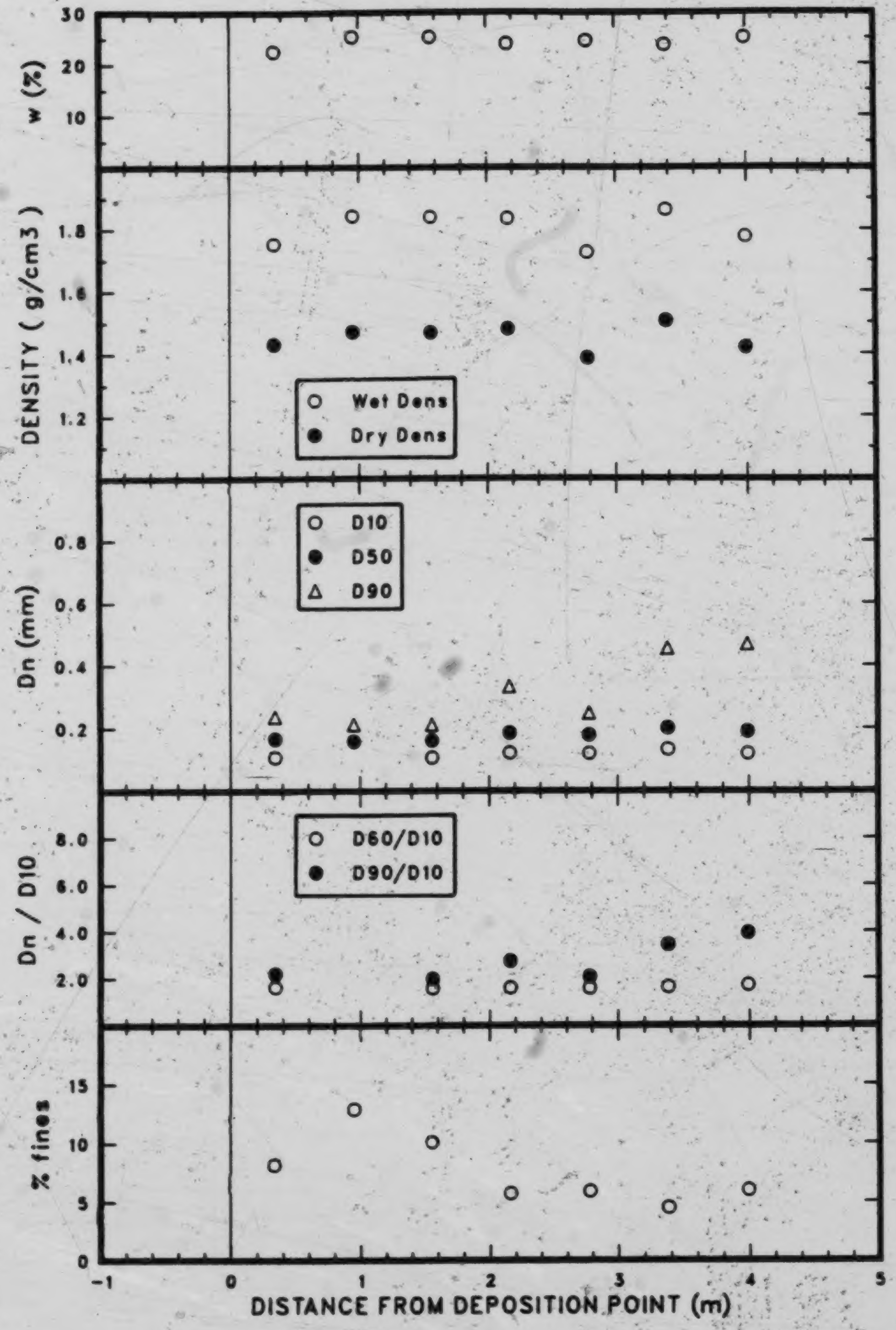


FIG. B.61 – Results of test TS28 (Cont.)

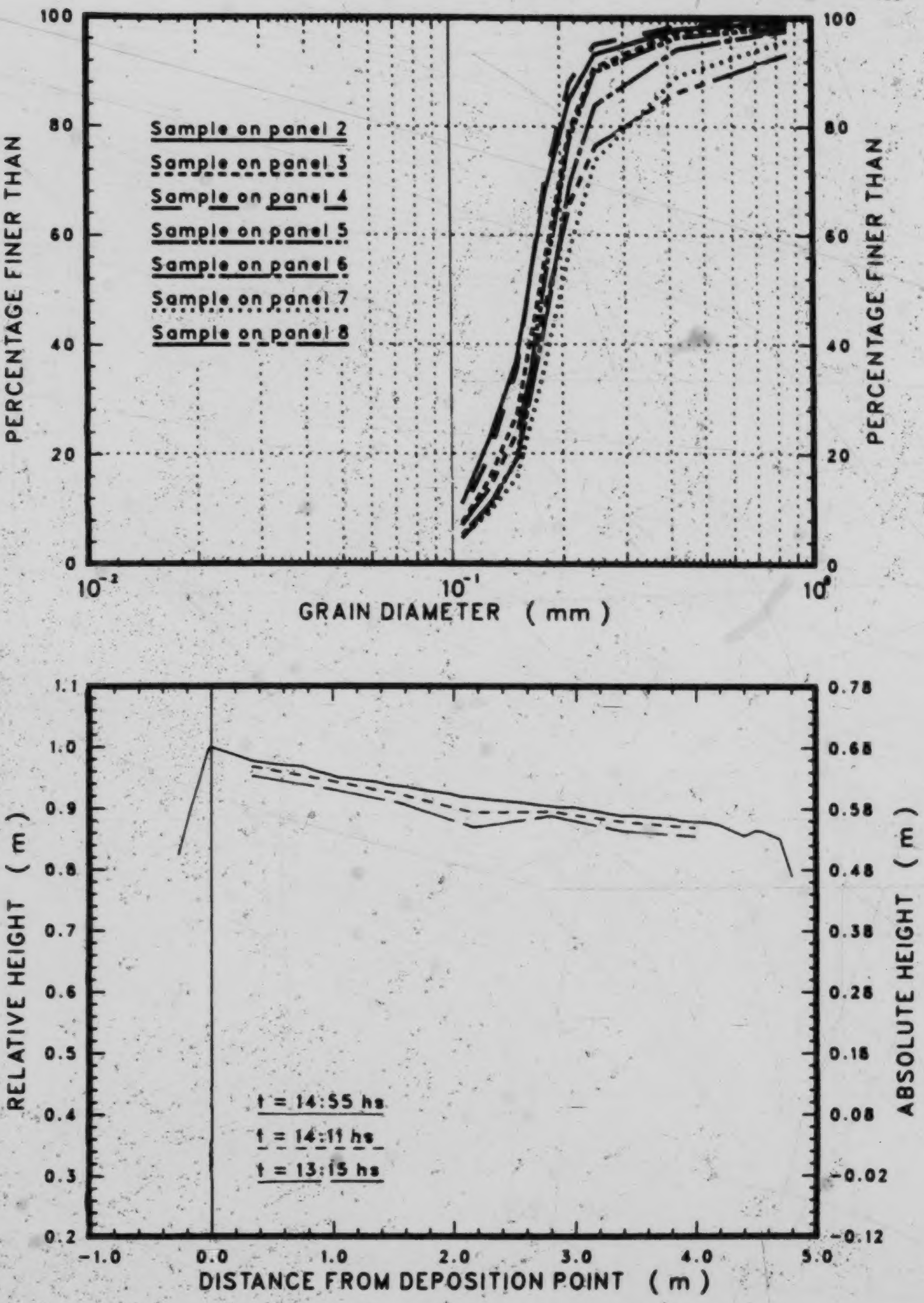


FIG. B.62 – Results of test TS29 (Qt = 15 l/min, Cw = 6 %)

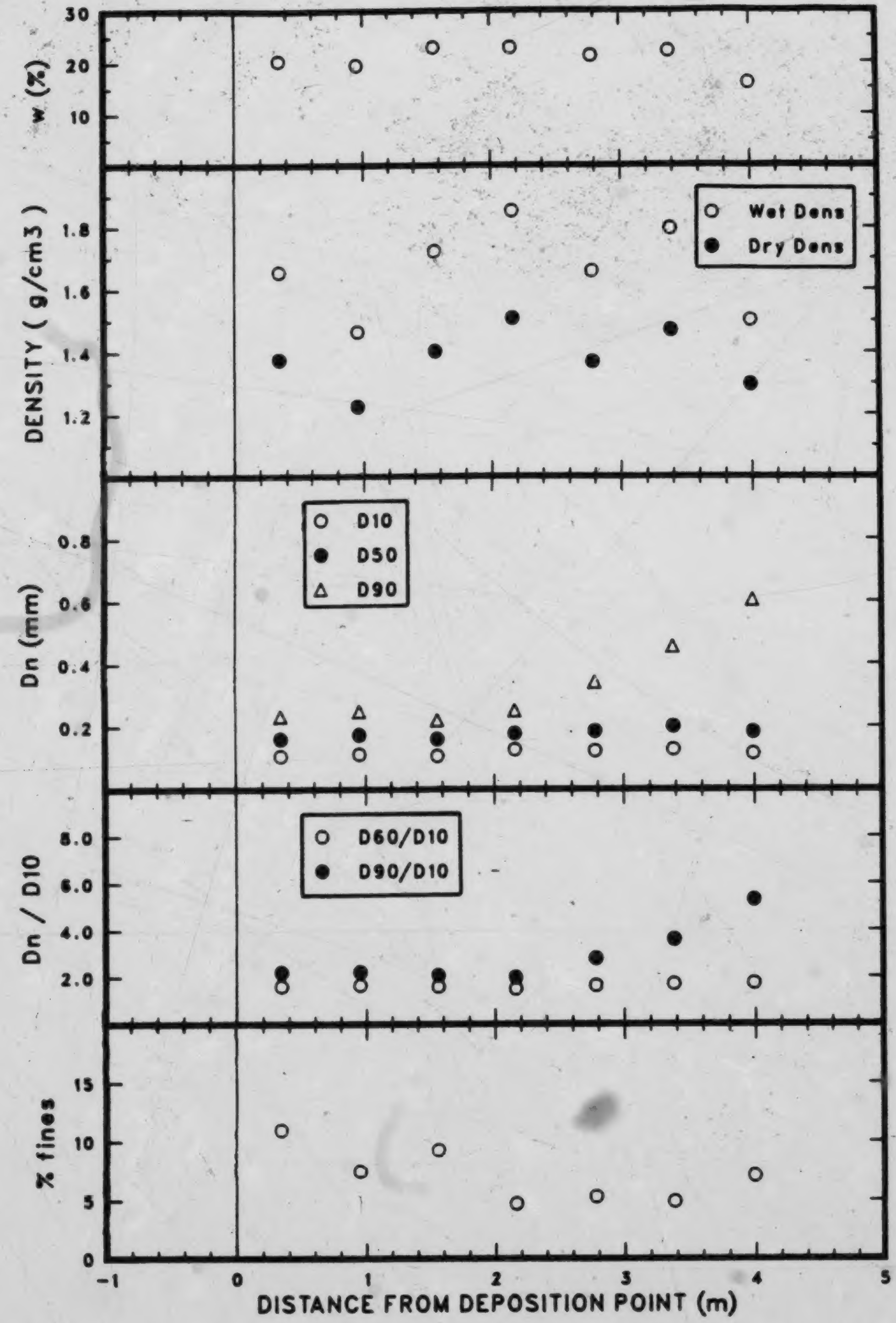


FIG. B.63 – Results of test TS29 (Cont.)

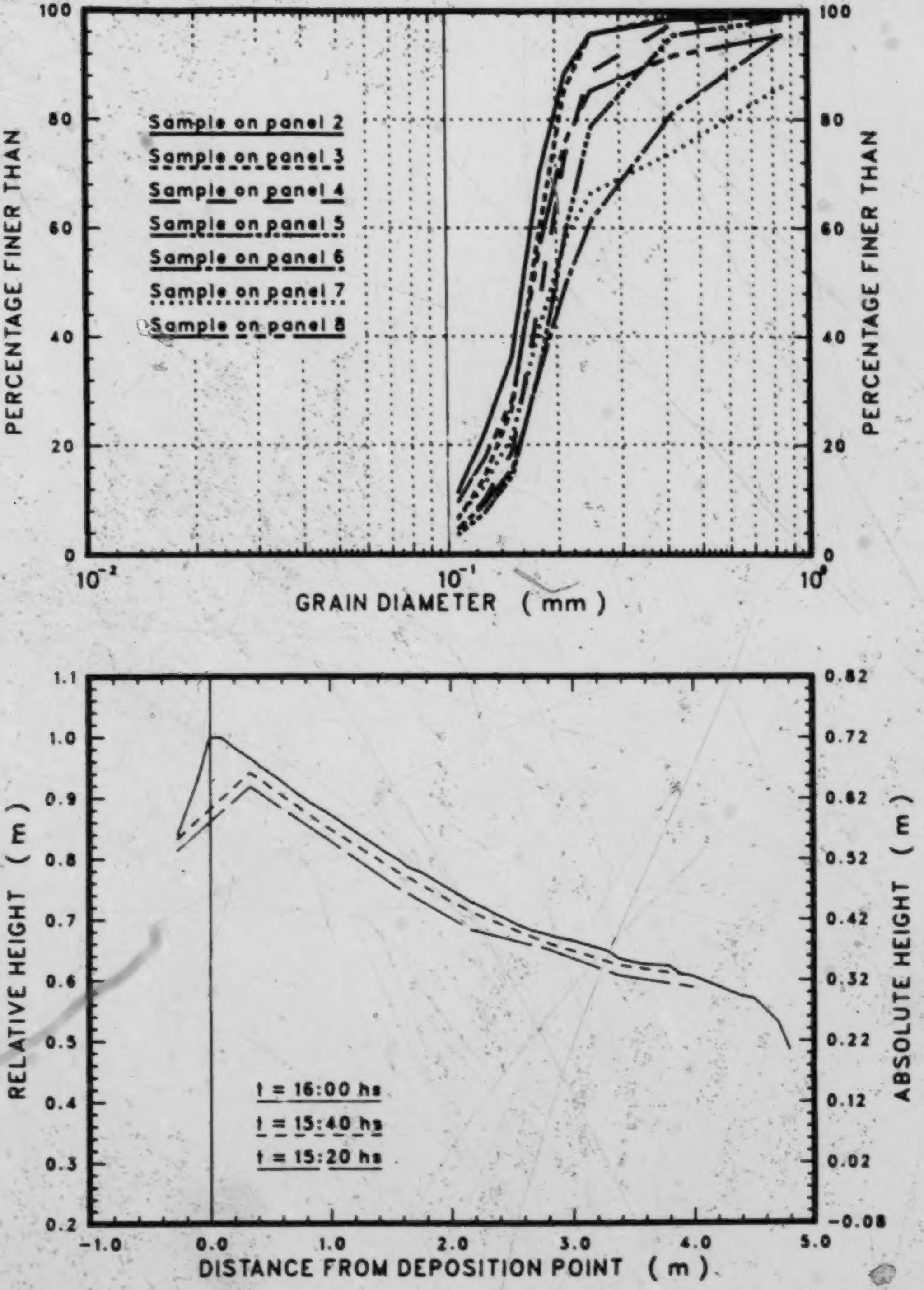


FIG. B.64 – Results of test TS30 (Qt = 5 l/min, Cw = 40 %)

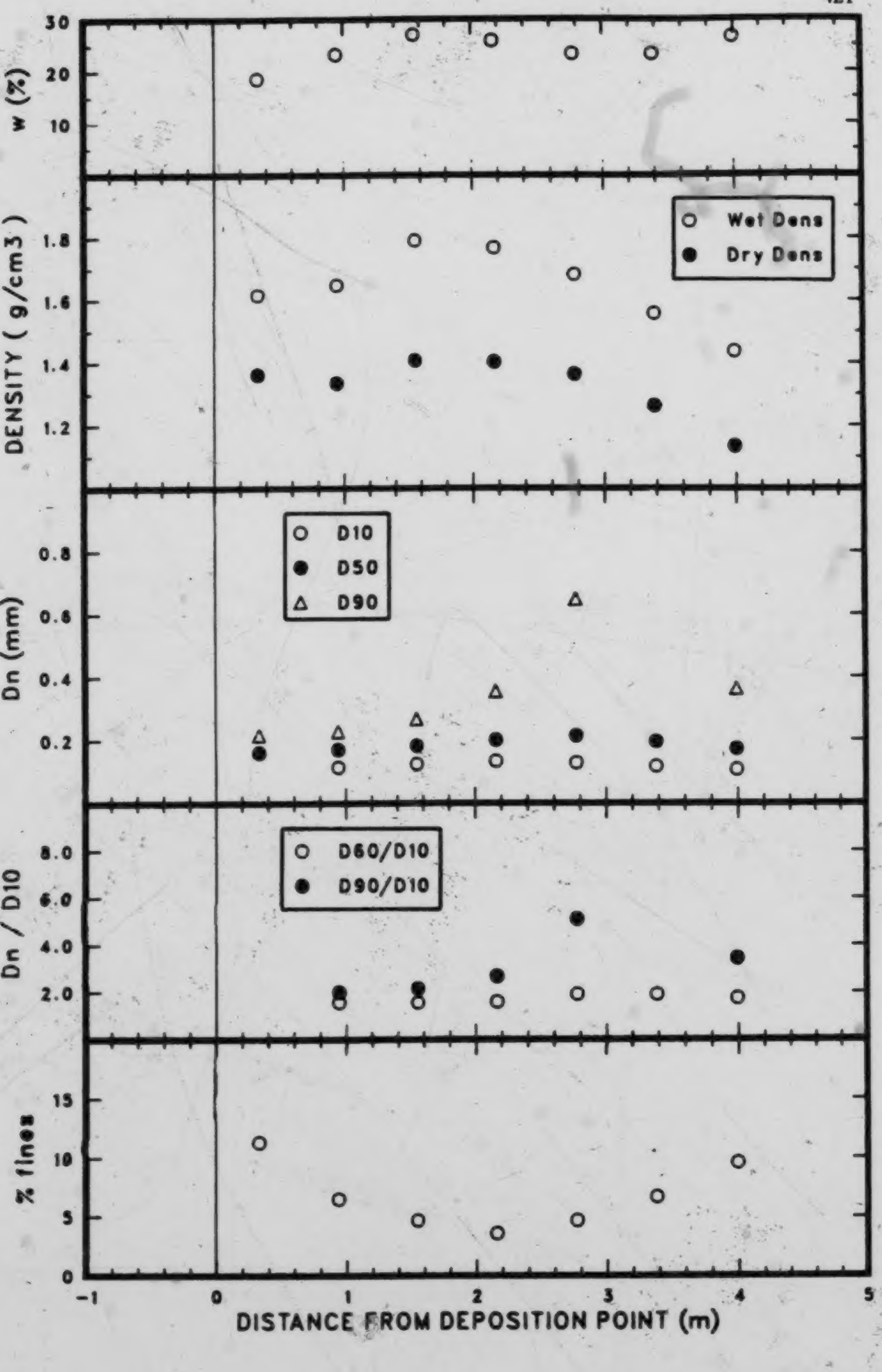
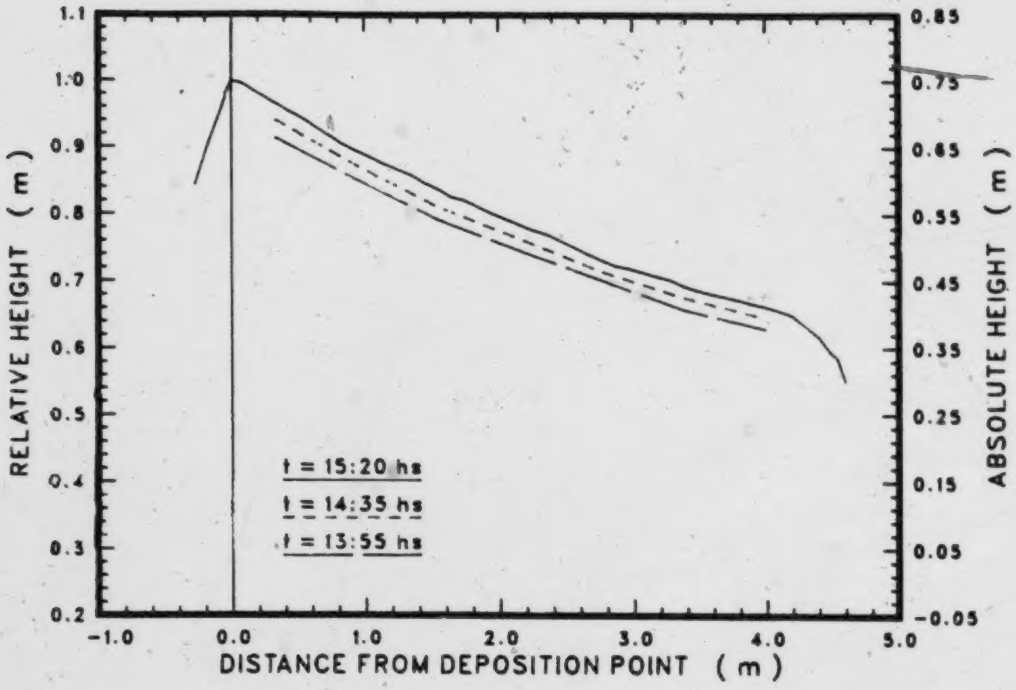


FIG. B.65 – Results of test TS30 (Cont.)



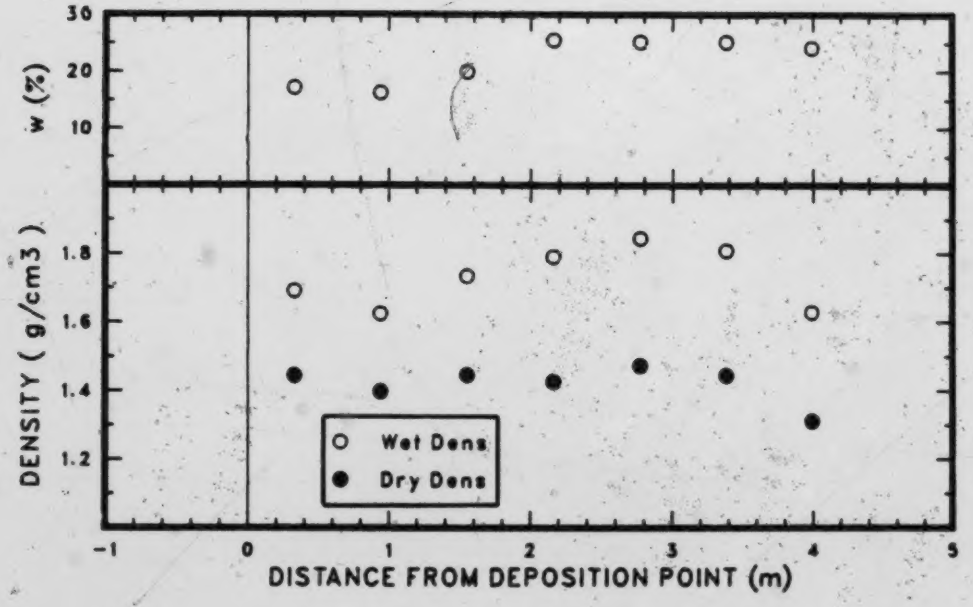


FIG. B.66 – Results of test TS31 (Qt = 5 l/min, Cw = 25 %)

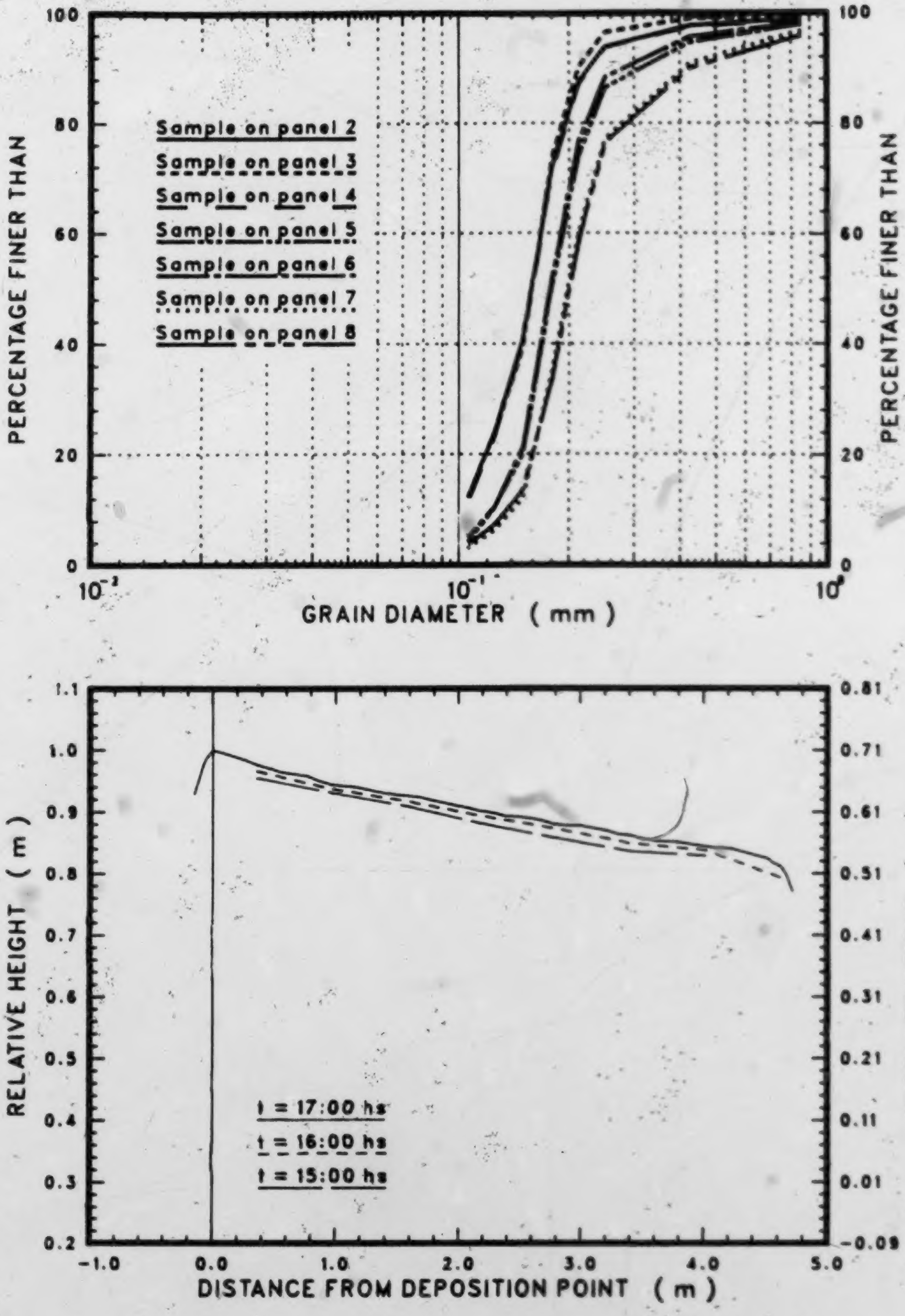


FIG. B.67 – Results of test TS32 (Qt = 10 l/min, Cw = 6 %)

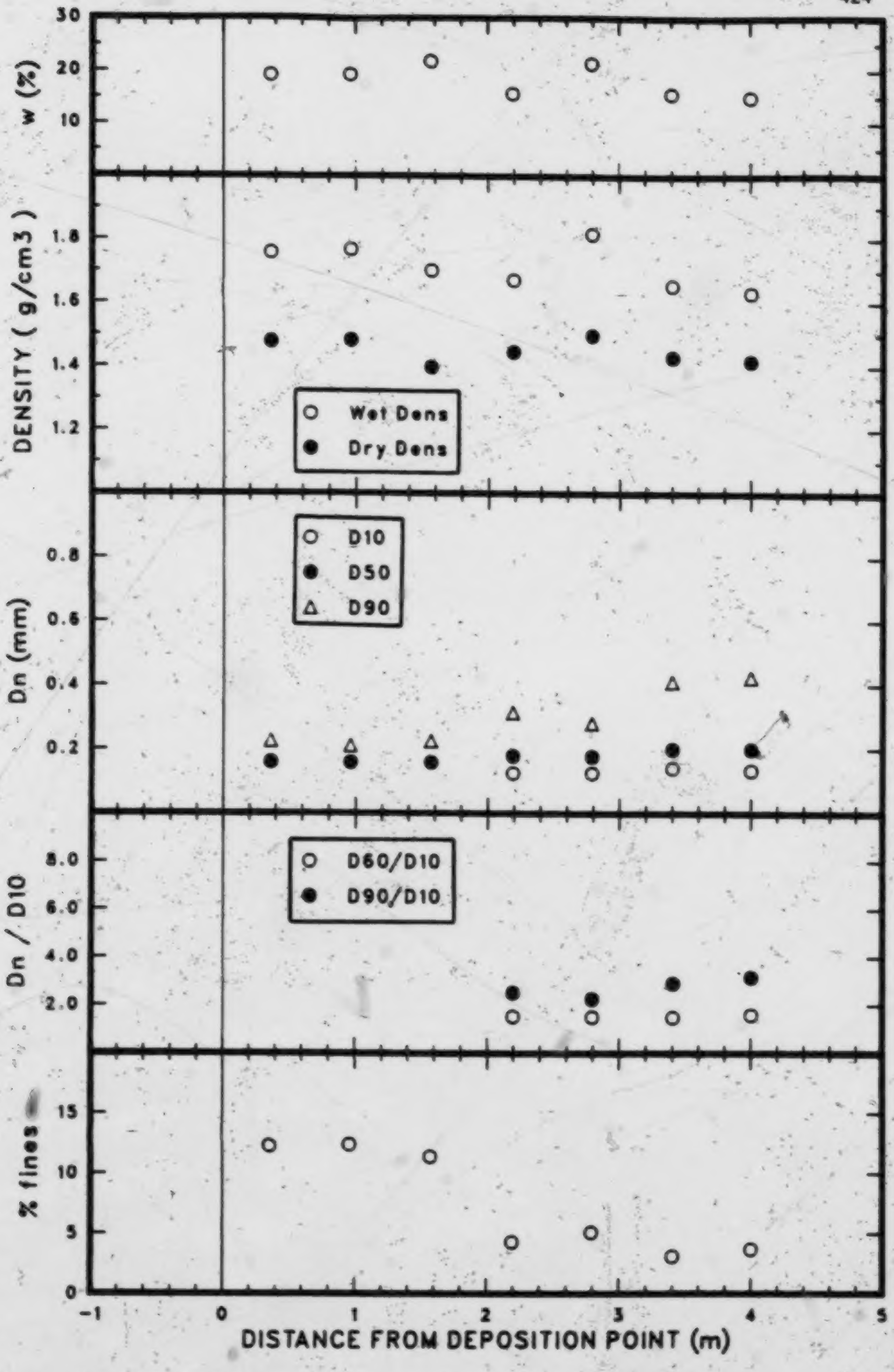


FIG. B.68 – Results of test TS32 (Cont.)

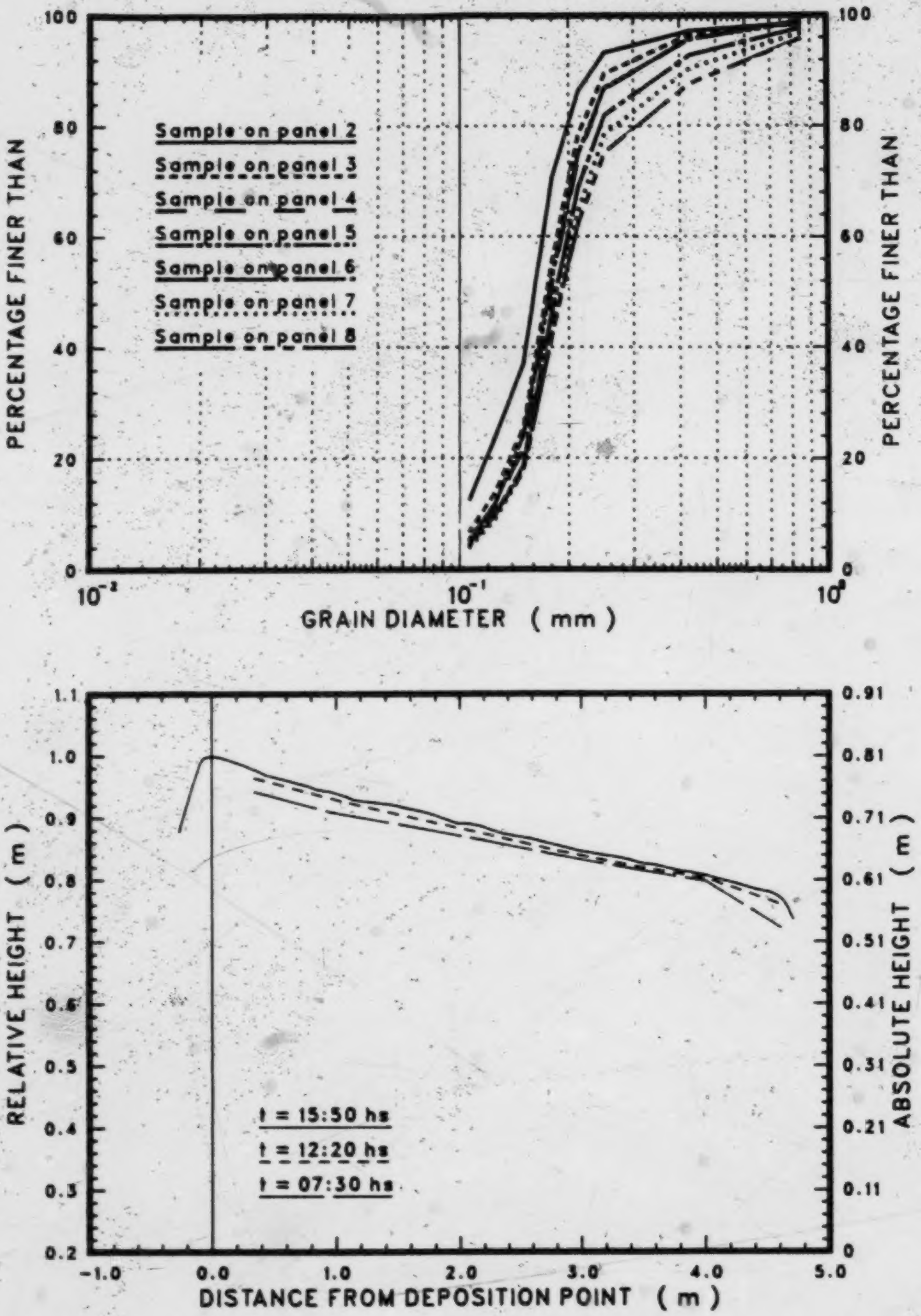


FIG. B.69 – Results of test TS33 (Qt = 5 l/min, Cw = 2 %)

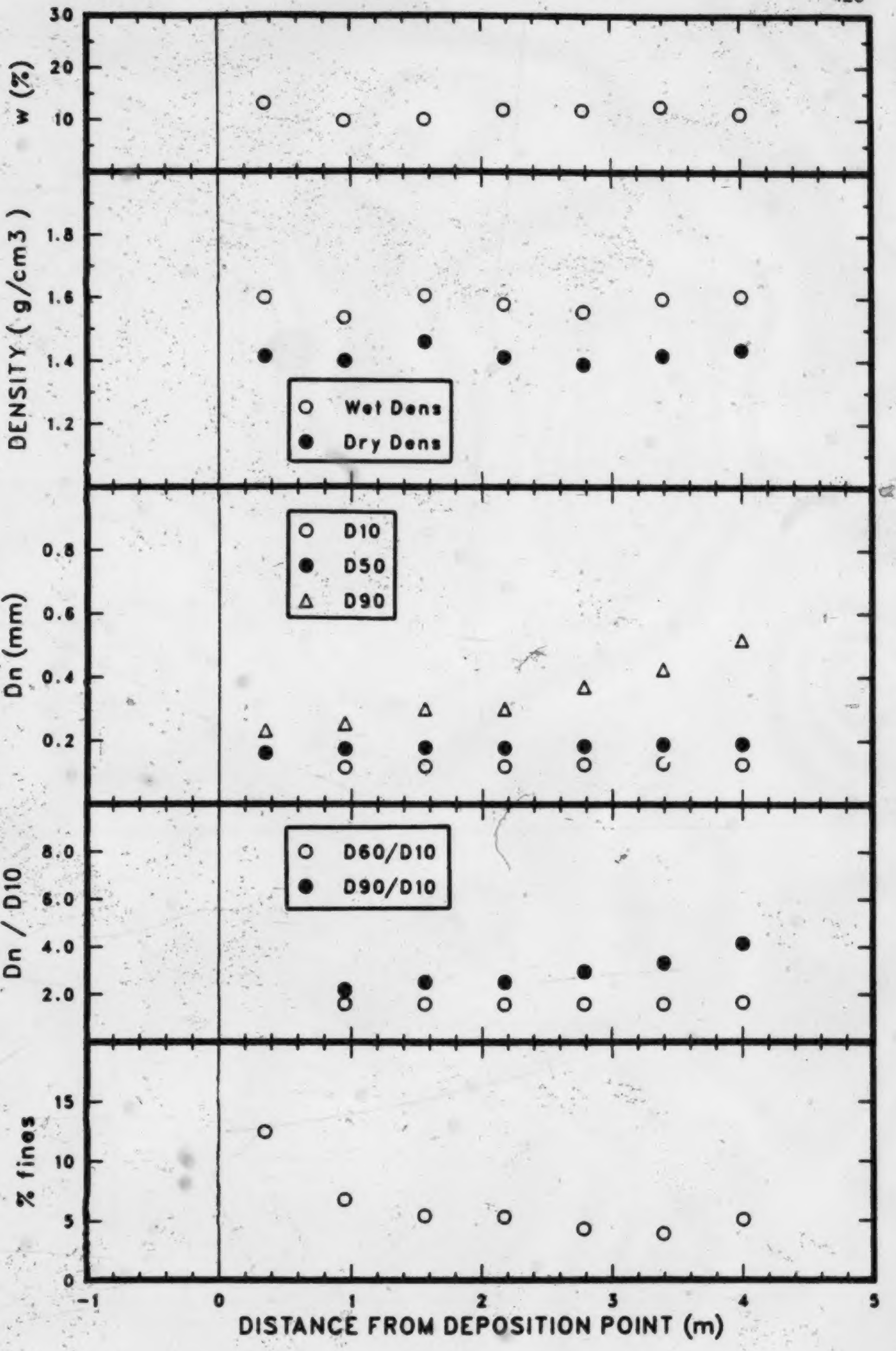
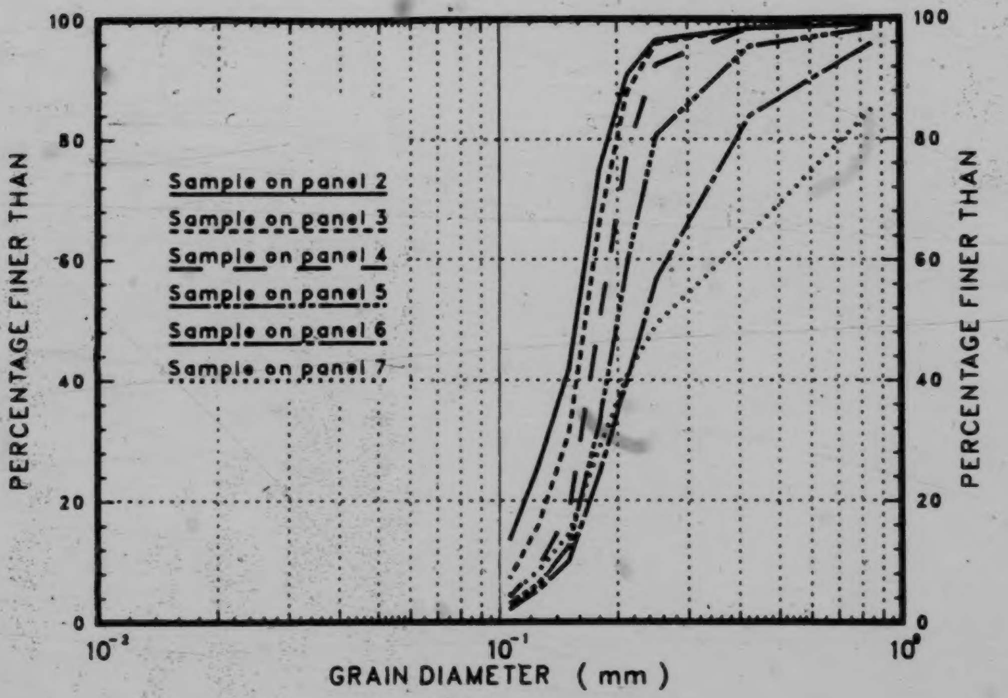


FIG. B.70 – Results of test TS33 (Cont.)



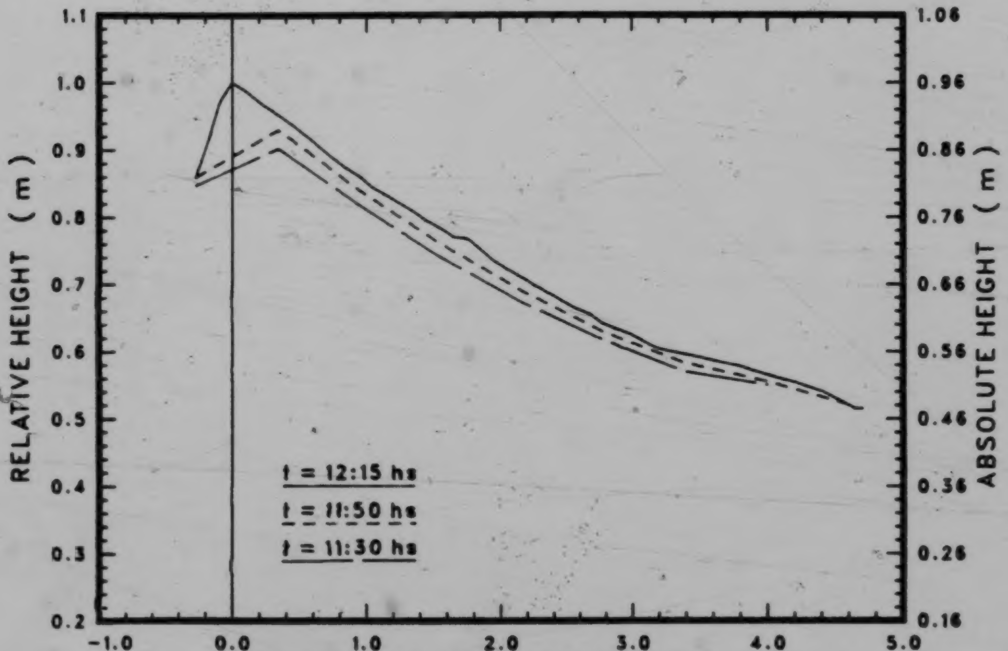


FIG. B.71 – Results of test TS34 (Qt = 5 l/min, Cw = 30 %)

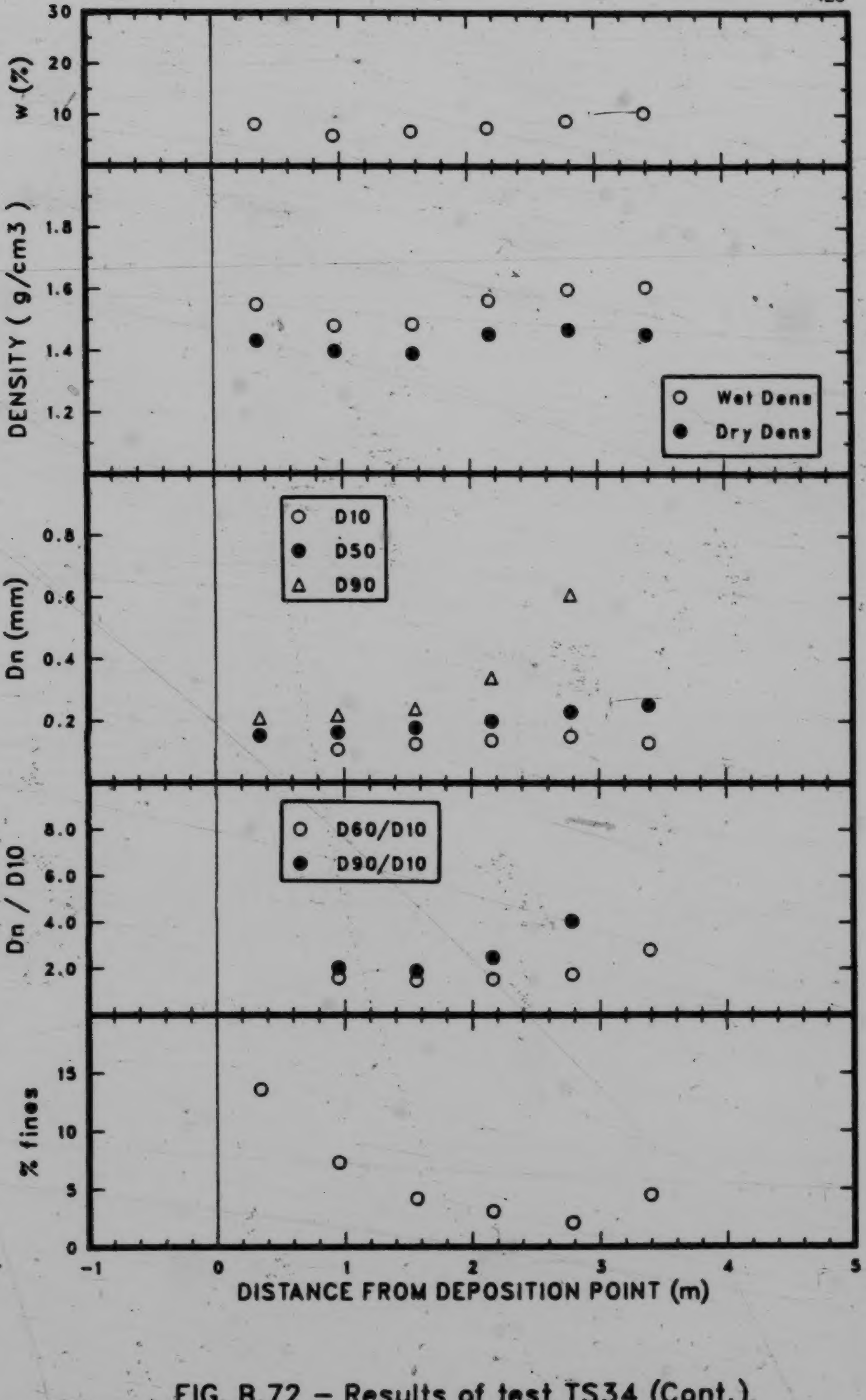


FIG. B.72 – Results of test TS34 (Cont.)

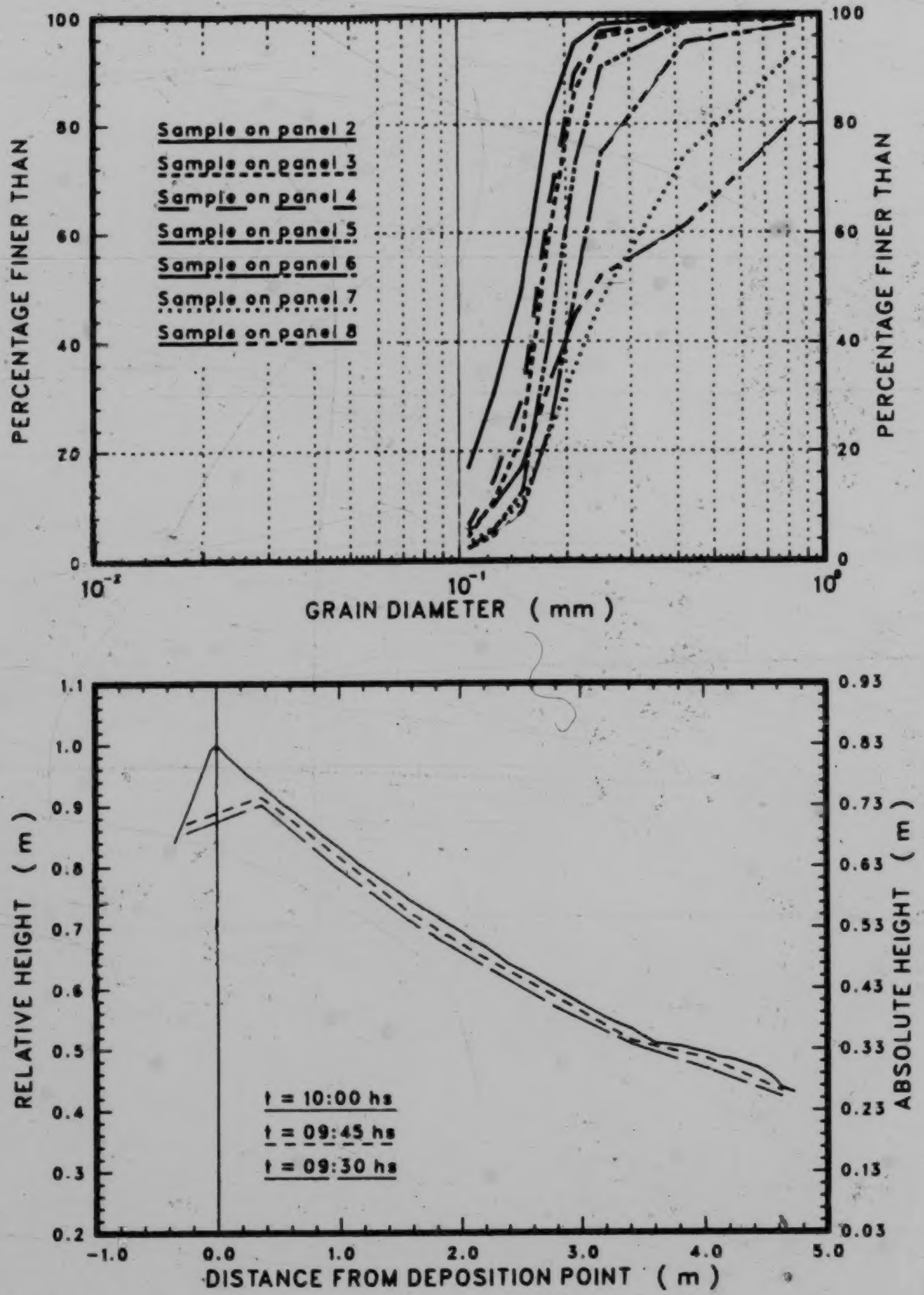


FIG. B.73 – Results of test TS35 (Qt = 5 l/min, Cw = 35 %)

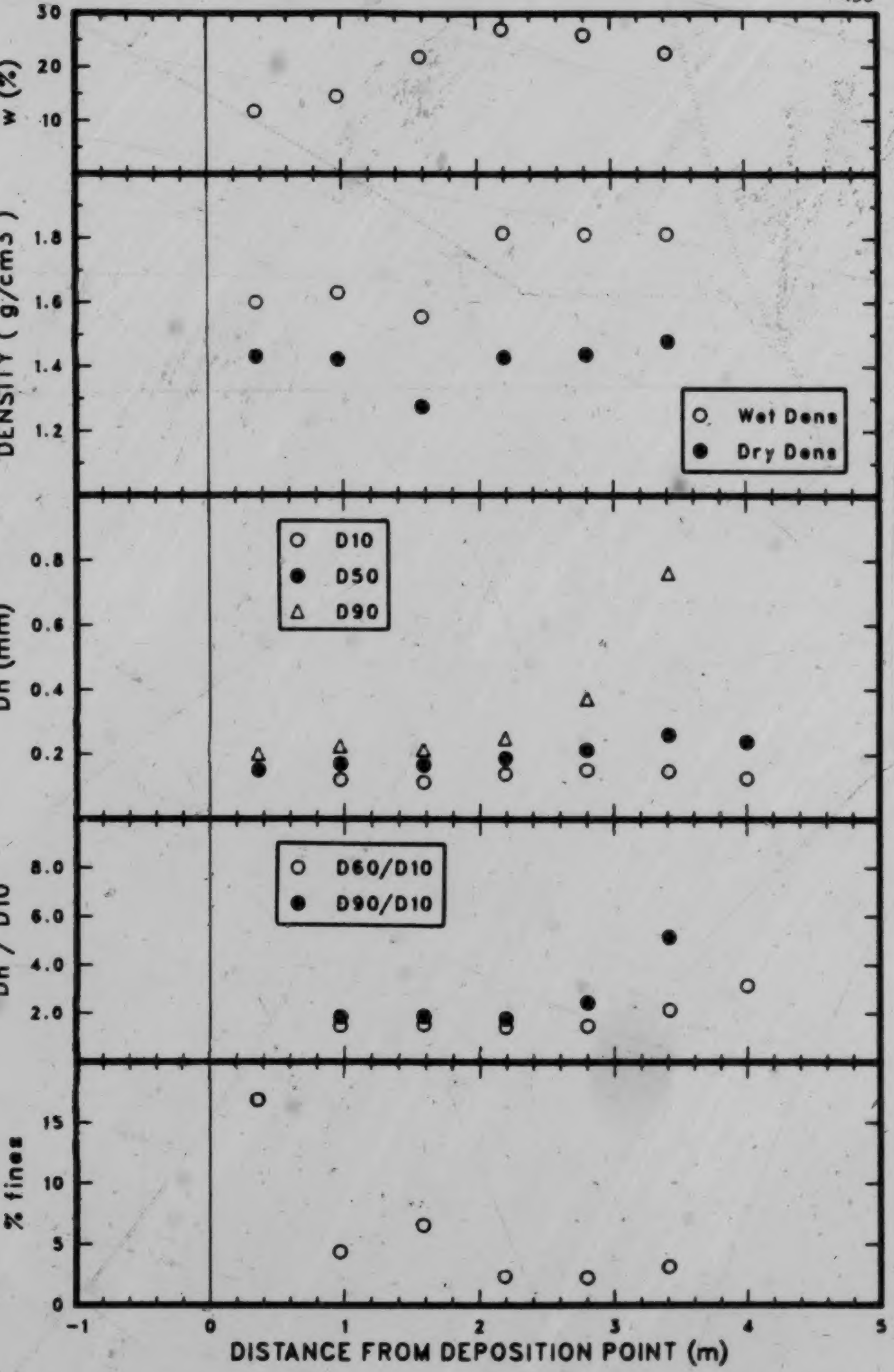


FIG. B.74 – Results of test TS35 (Cont.)

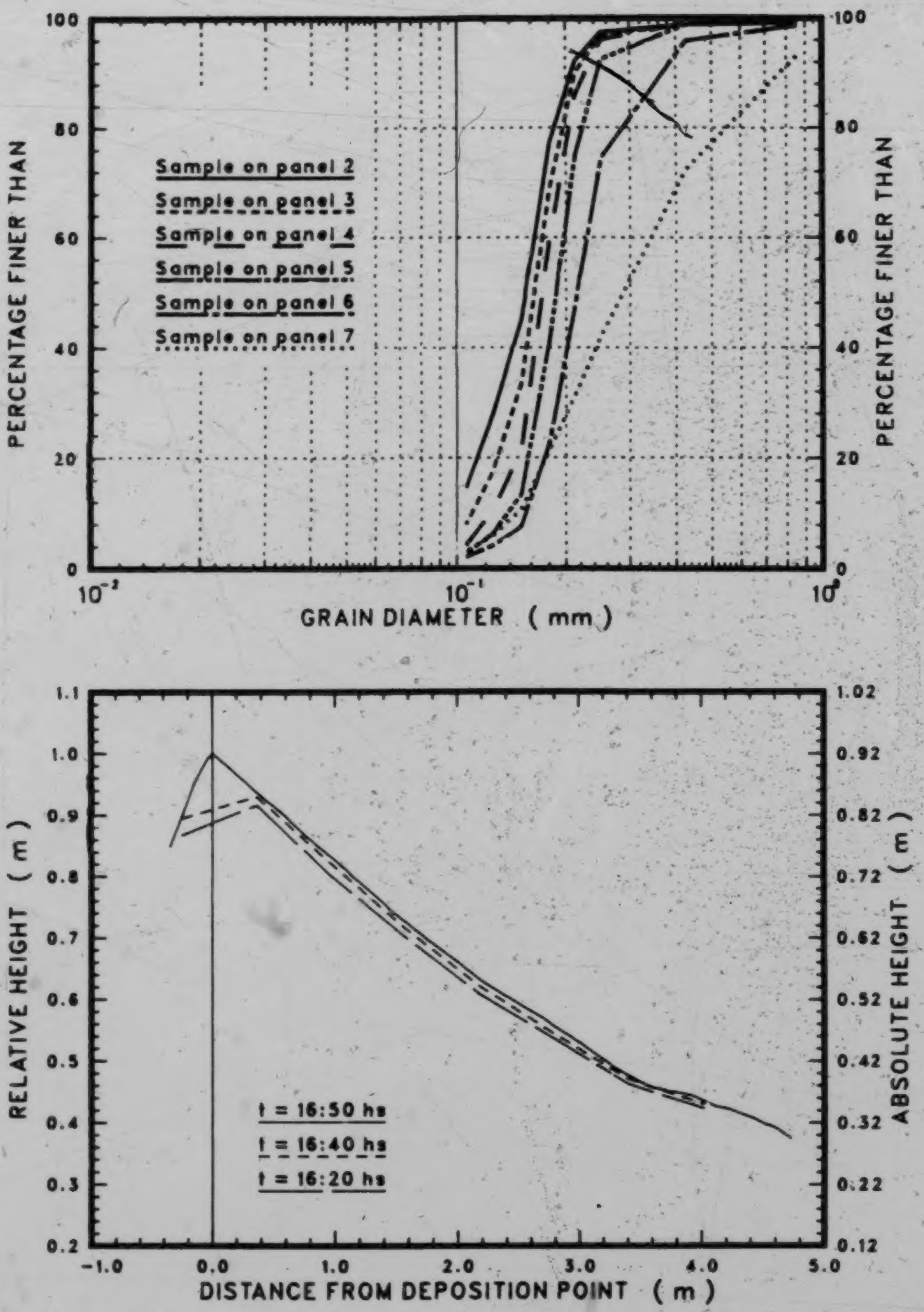


FIG. B.75 – Results of test TS36 (Qt = 5 l/min, Cw = 40 %)

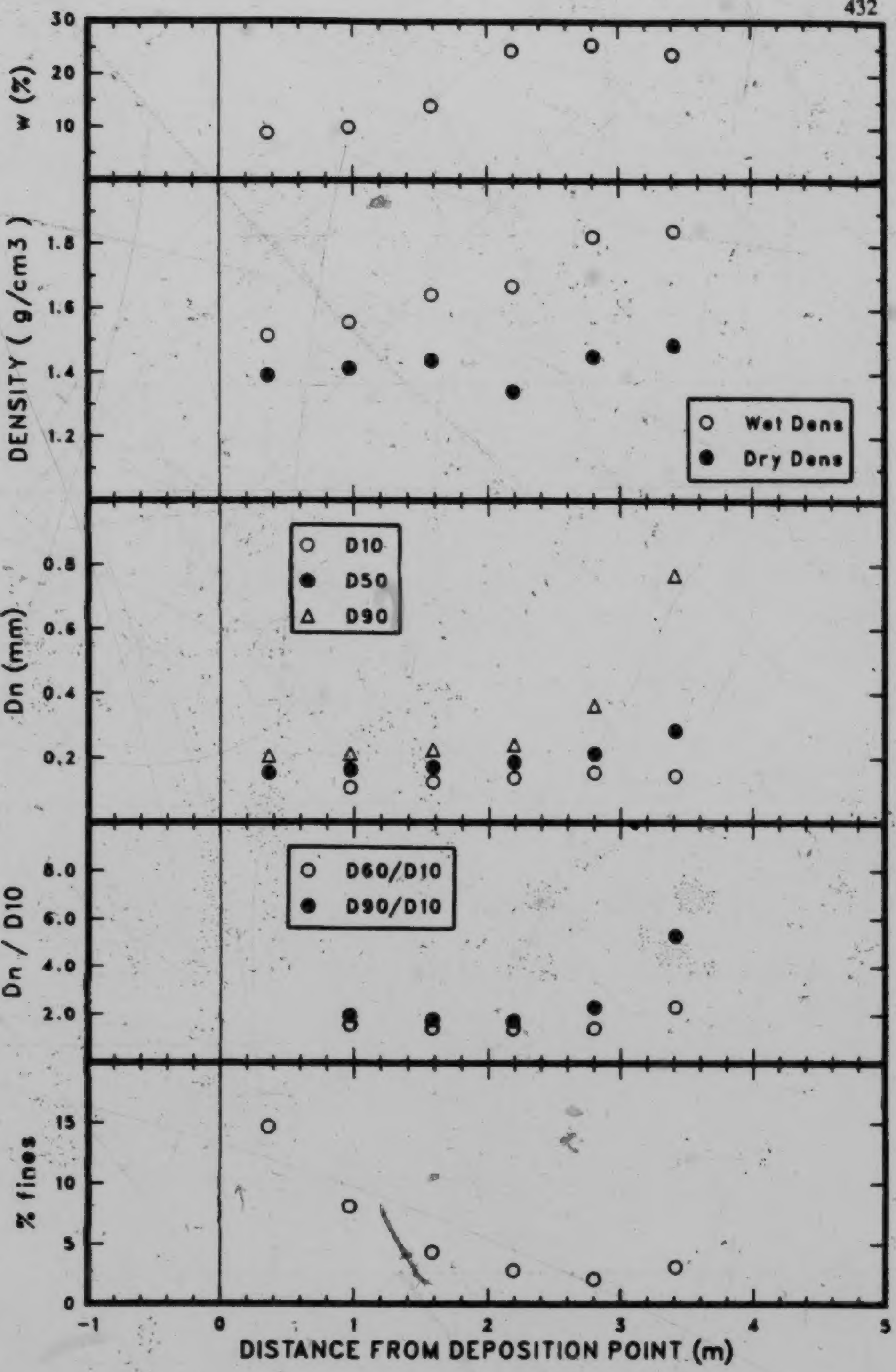
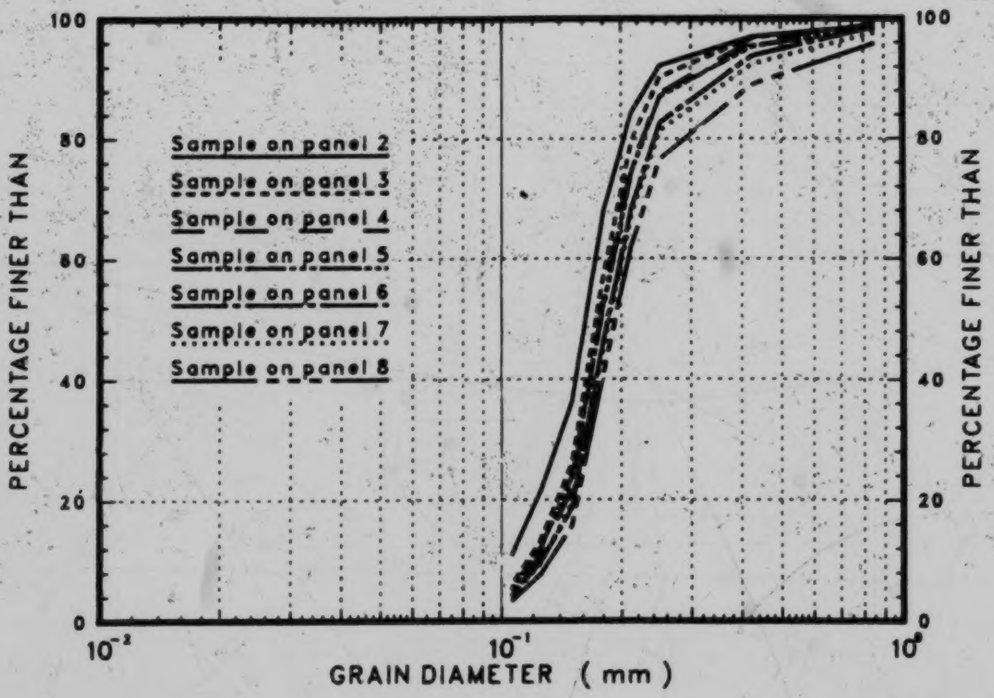


FIG. B.76 – Results of test TS36 (Cont.)



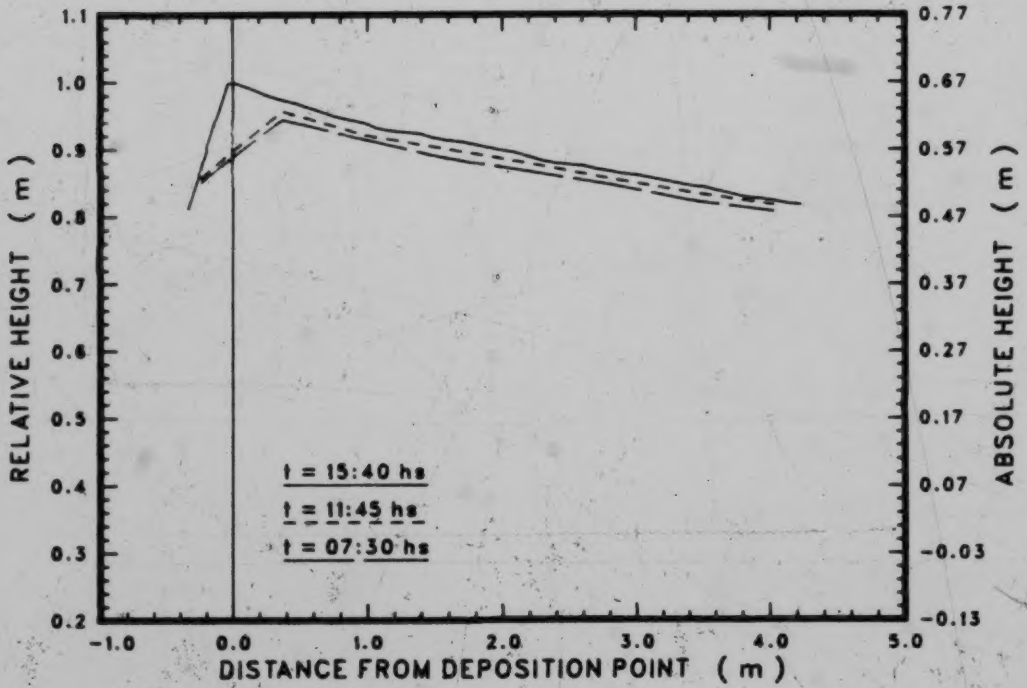


FIG. B.77 – Results of test TS37 (Qt = 5 l/min, Cw = 2 %)

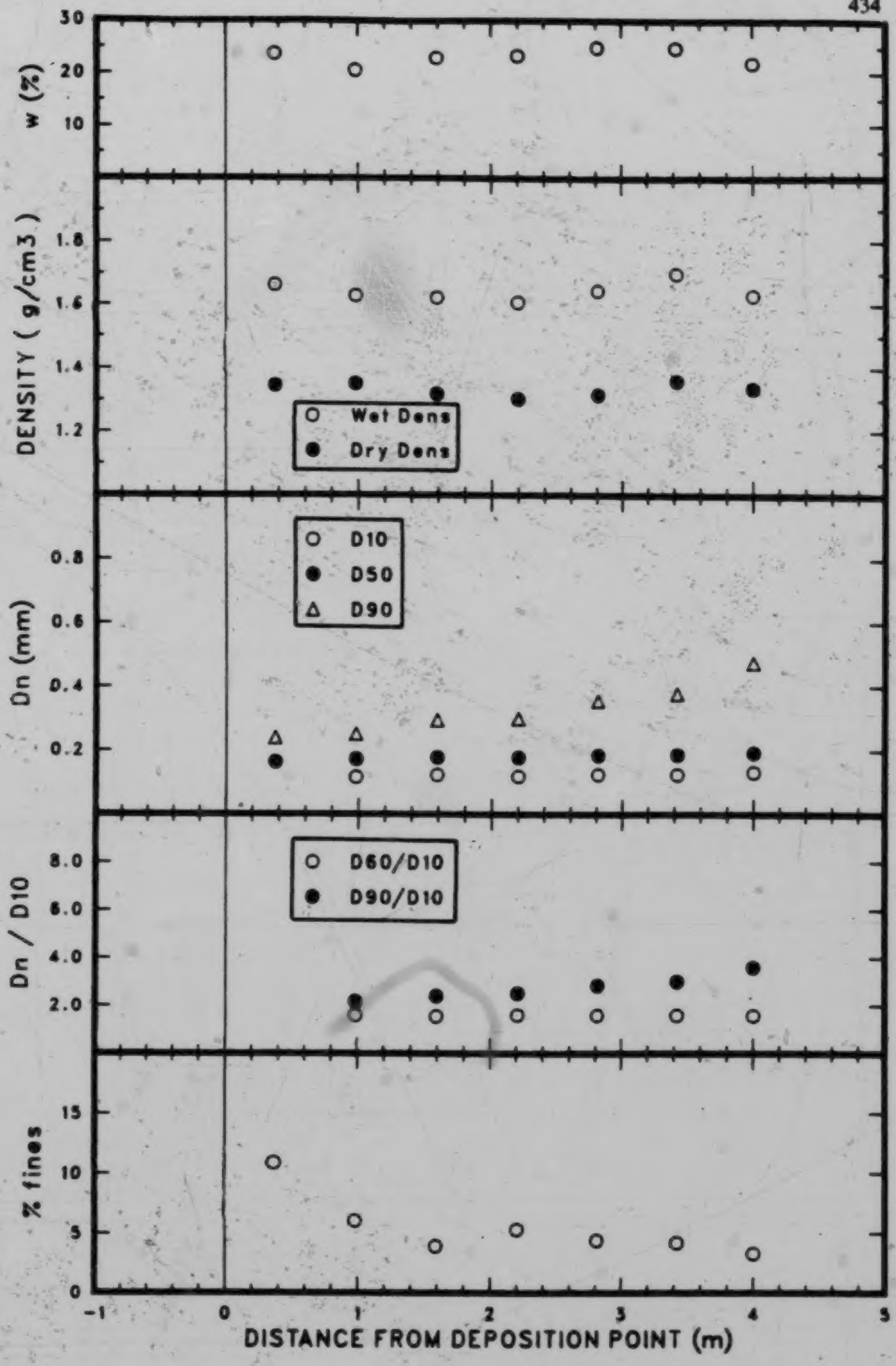


FIG. B.78 – Results of test TS37 (Cont.)

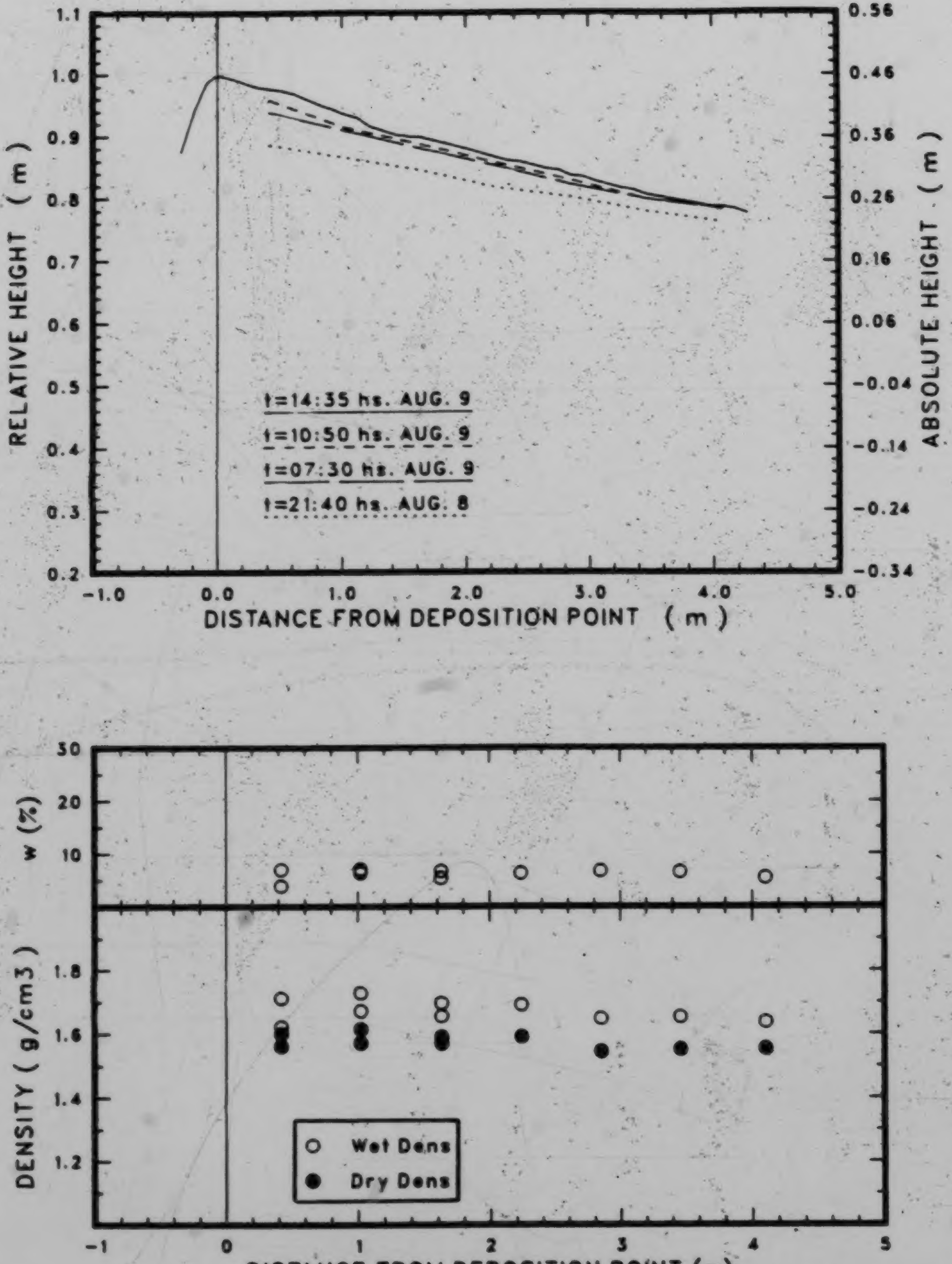
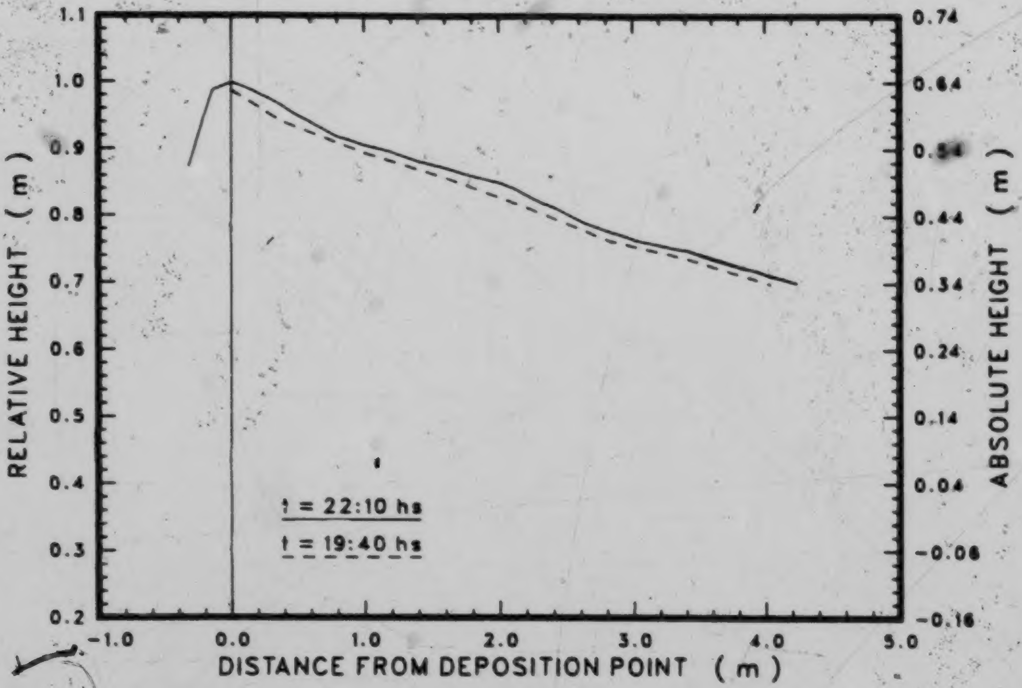


FIG. B.79 – Results of test KS1 (Qt = 5 l/min, Cw = 2 %)



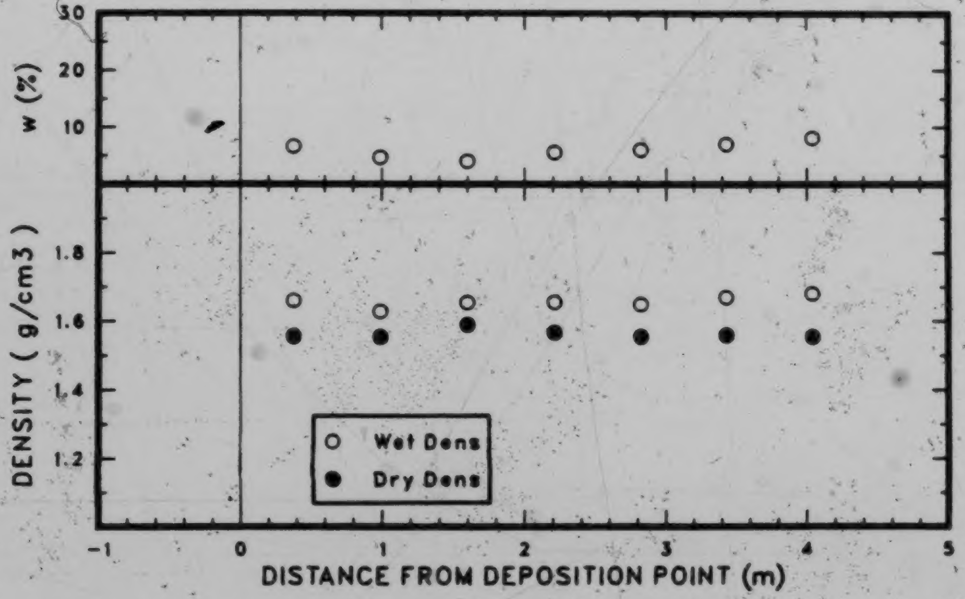
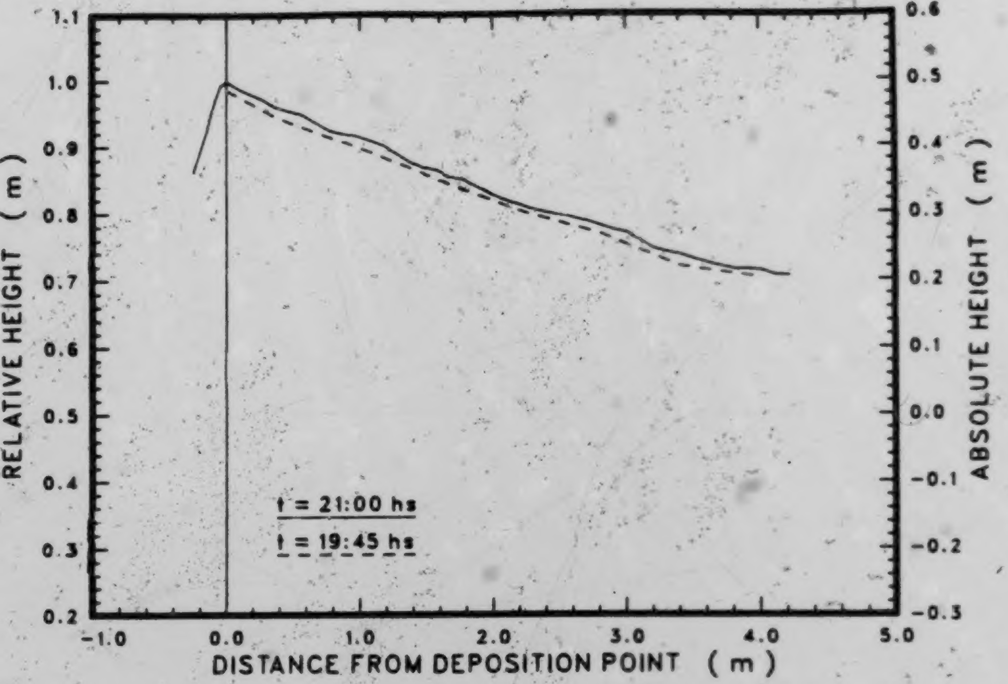


FIG. B.80 – Results of test KS3 (Qt = 5 l/min, Cw = 6 %)



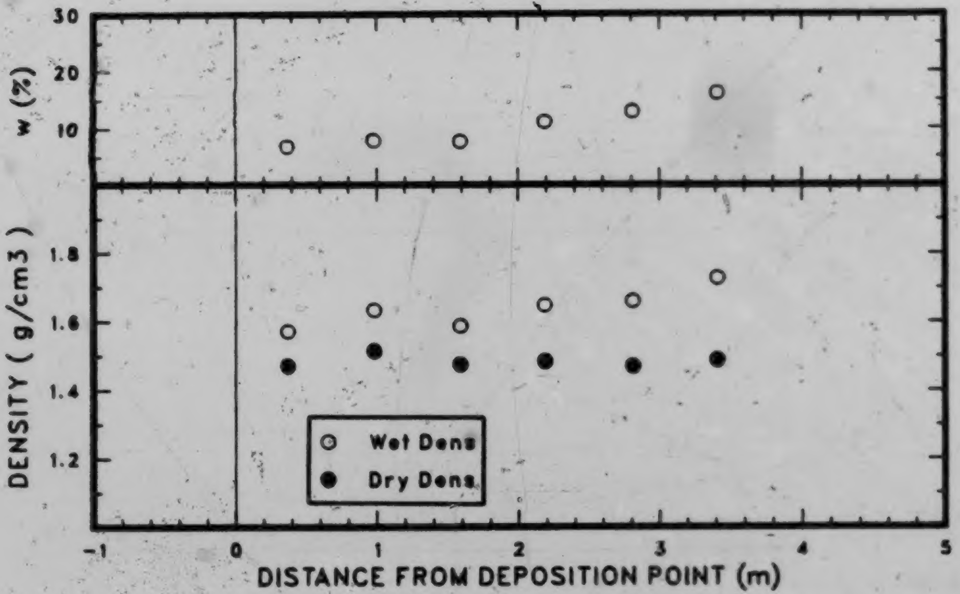


FIG. B.81 – Results of test KS4 (Qt = 5 l/min, Cw = 10 %)

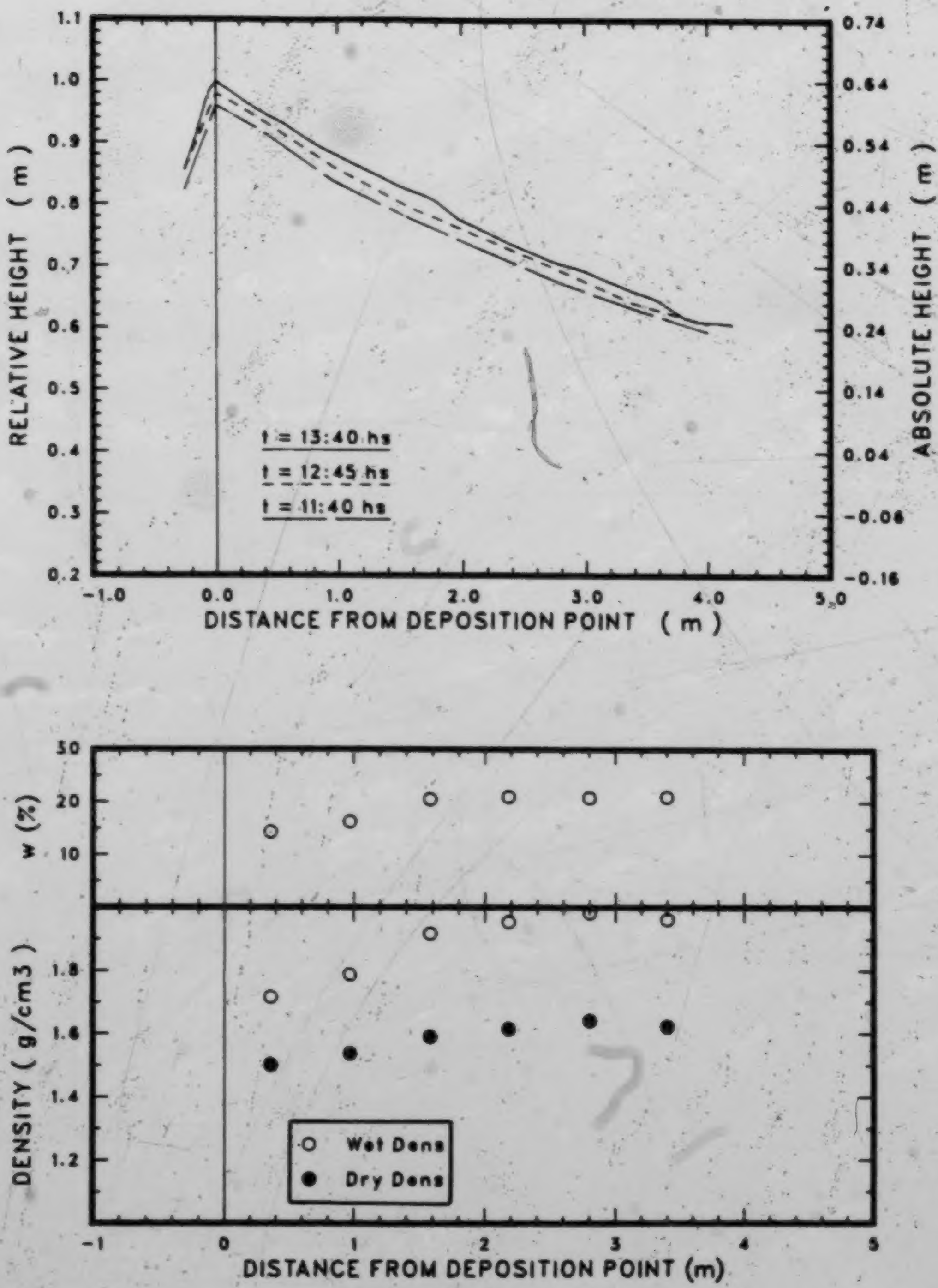


FIG. B.82 – Results of test KS5 (Qt = 5 l/min, Cw = 15 %)

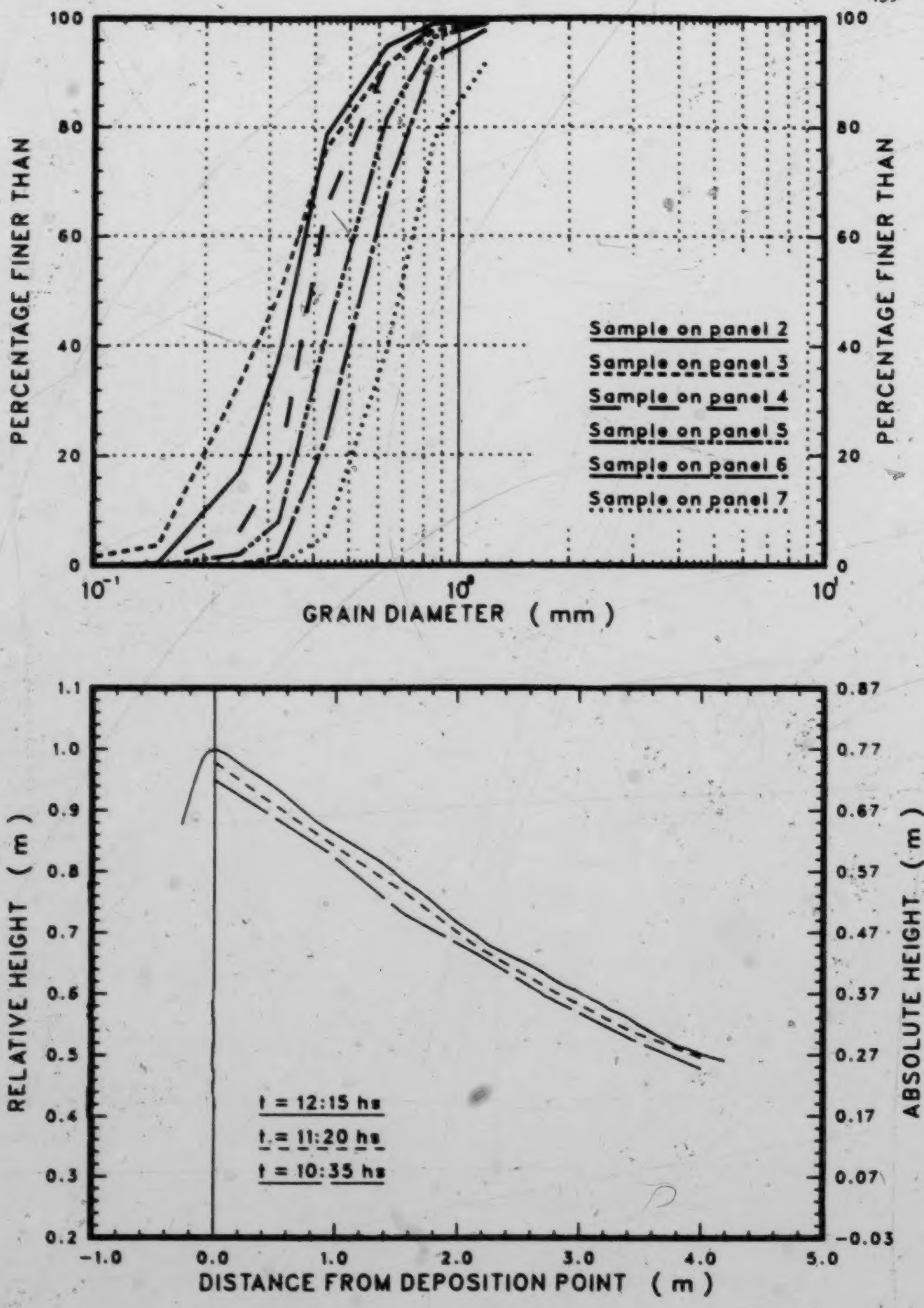


FIG. B.83 – Results of test KS6 (Qt = 5 l/min, Cw = 20 %)

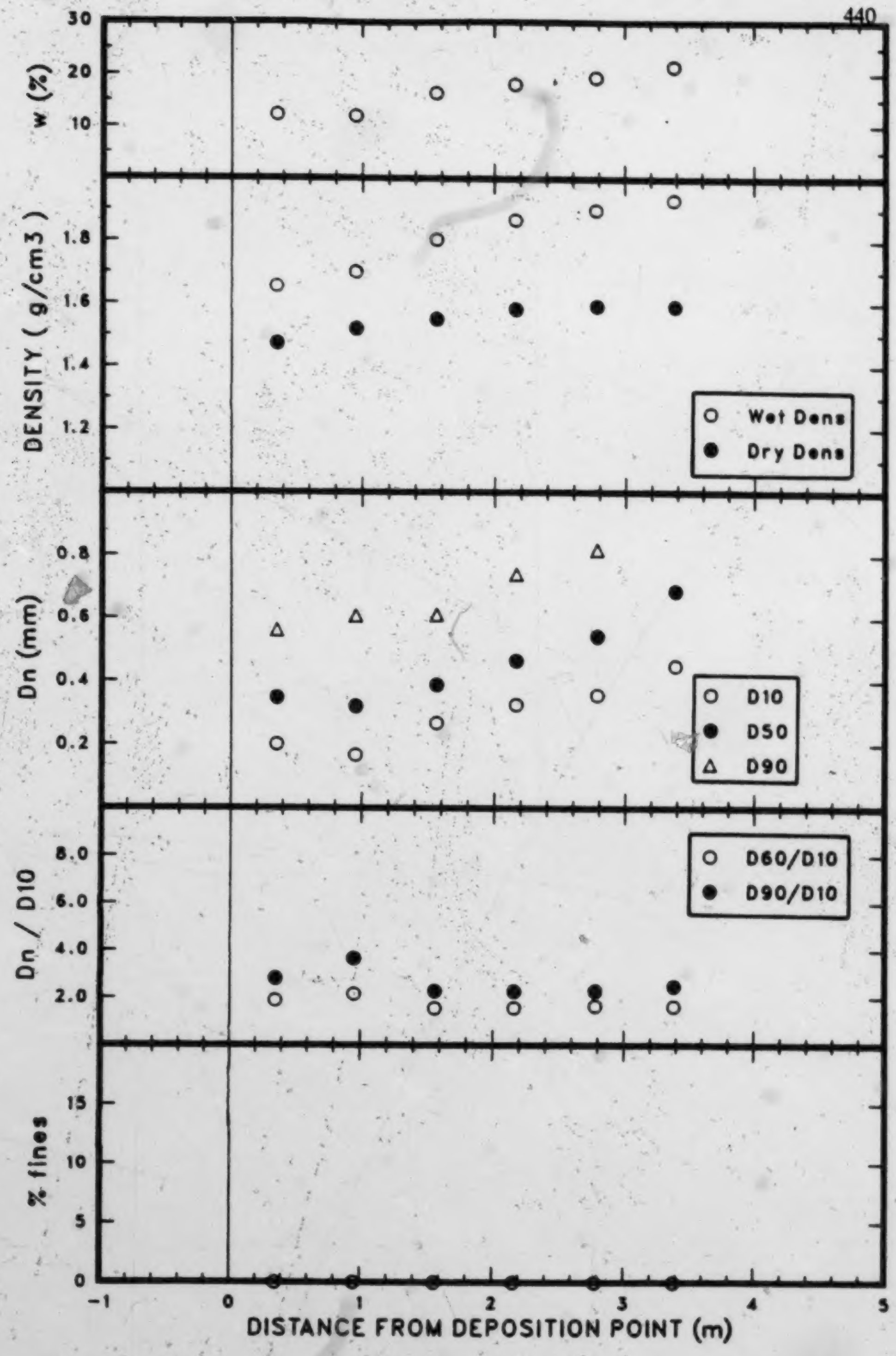
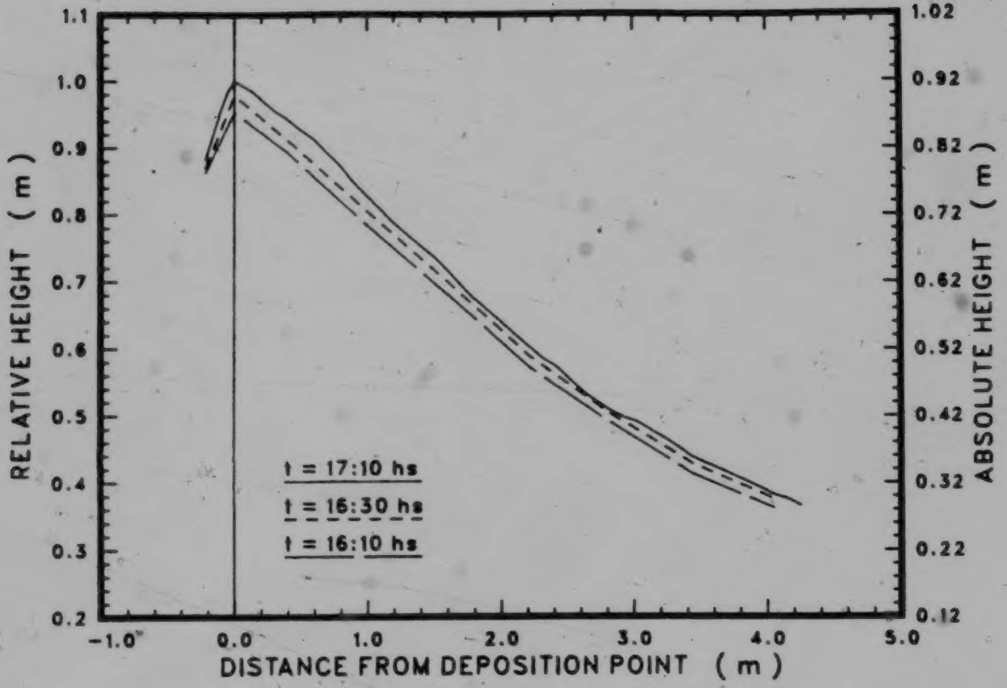


FIG. B.84 – Results of test KS6 (Cont.)



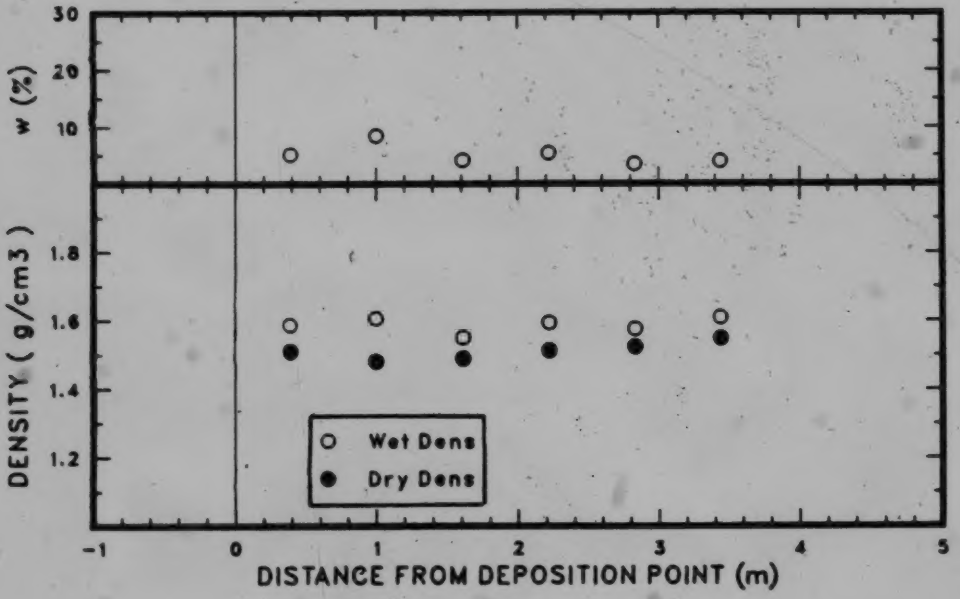


FIG. B.85 – Results of test KS7 (Qt = 5 l/min, Cw = 25 %)

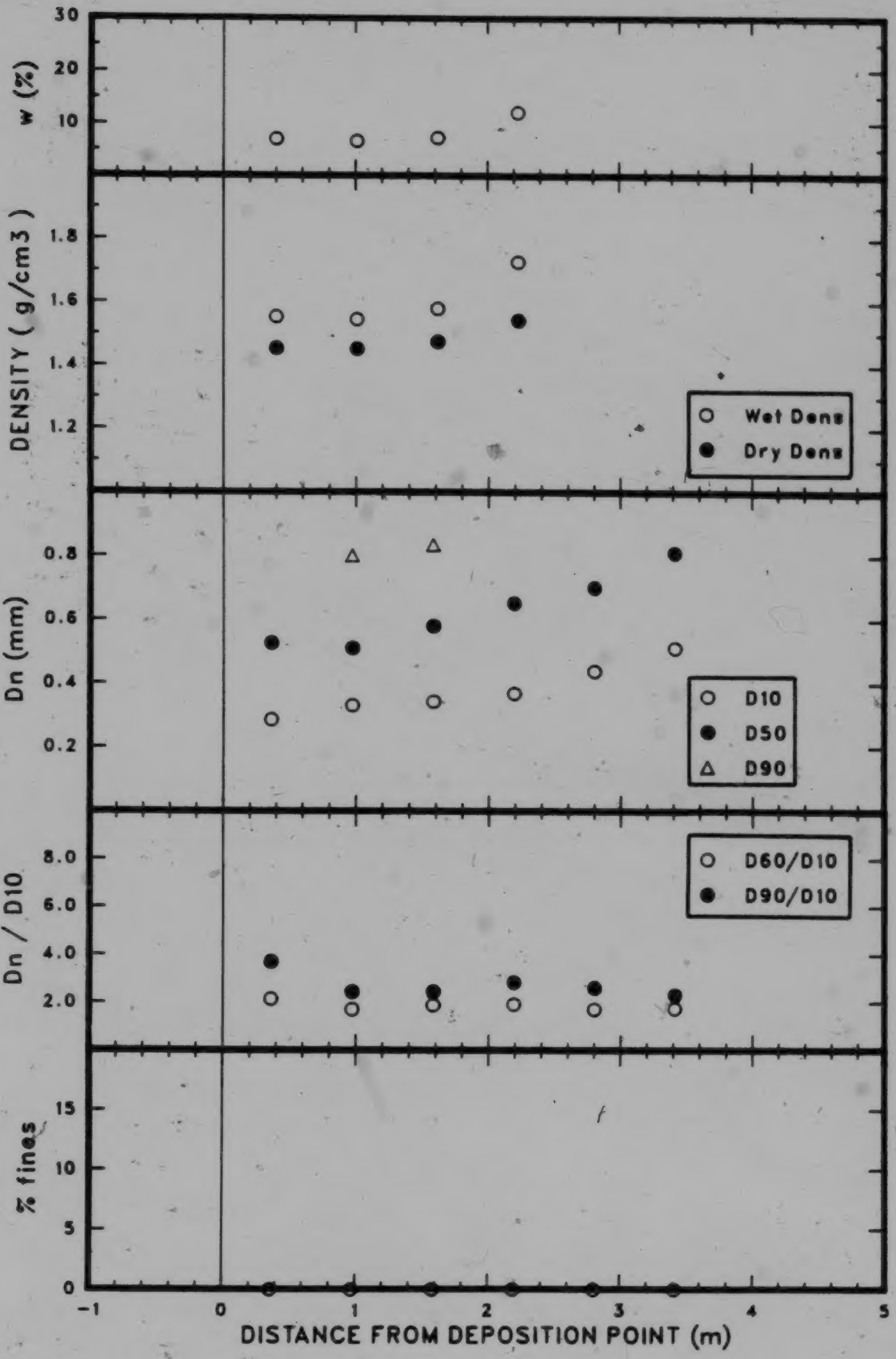


FIG. B.86 – Results of test KS8 (Cont.)

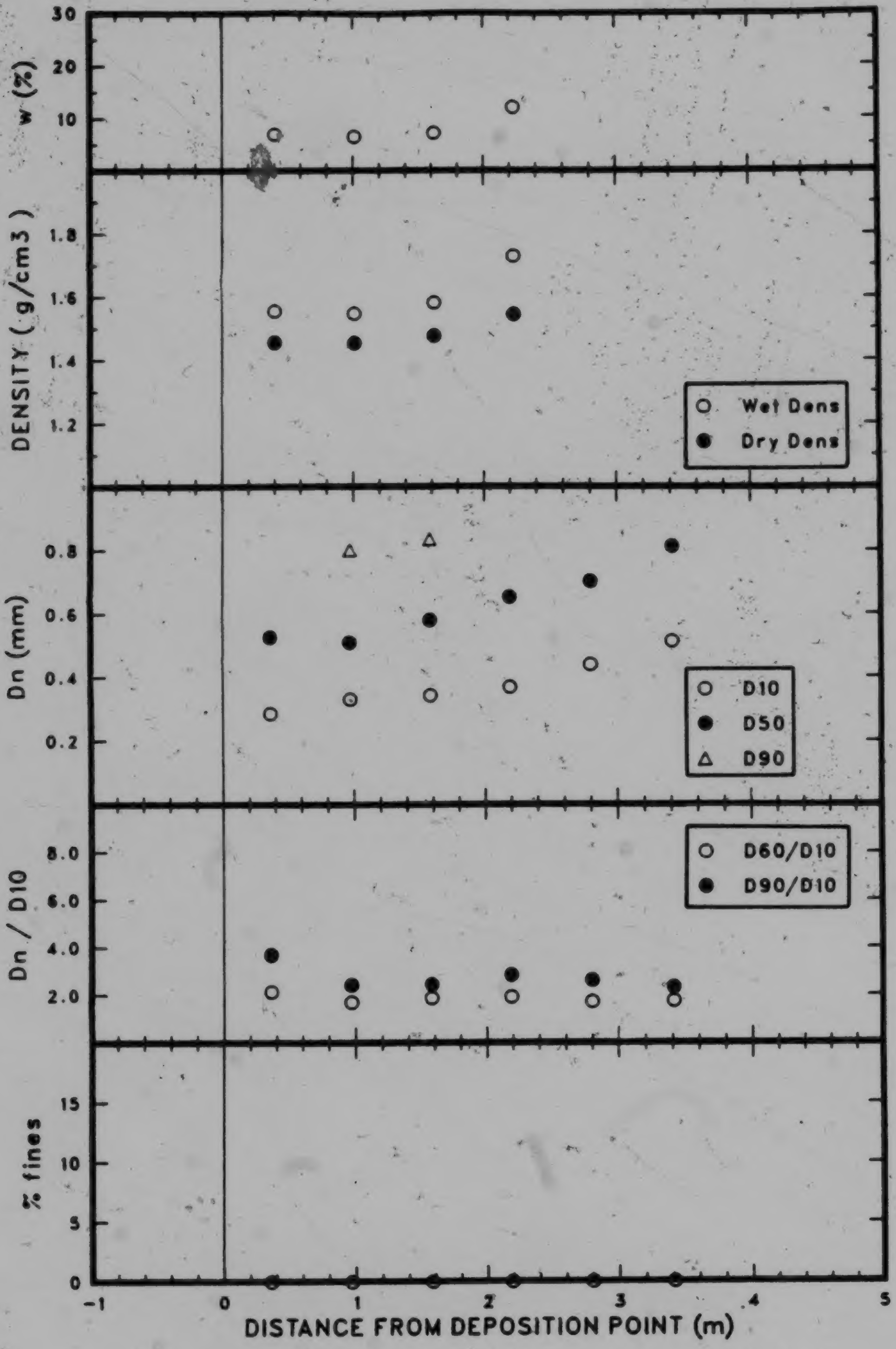


FIG. B.87 – Results of test KS8 (Cont.)

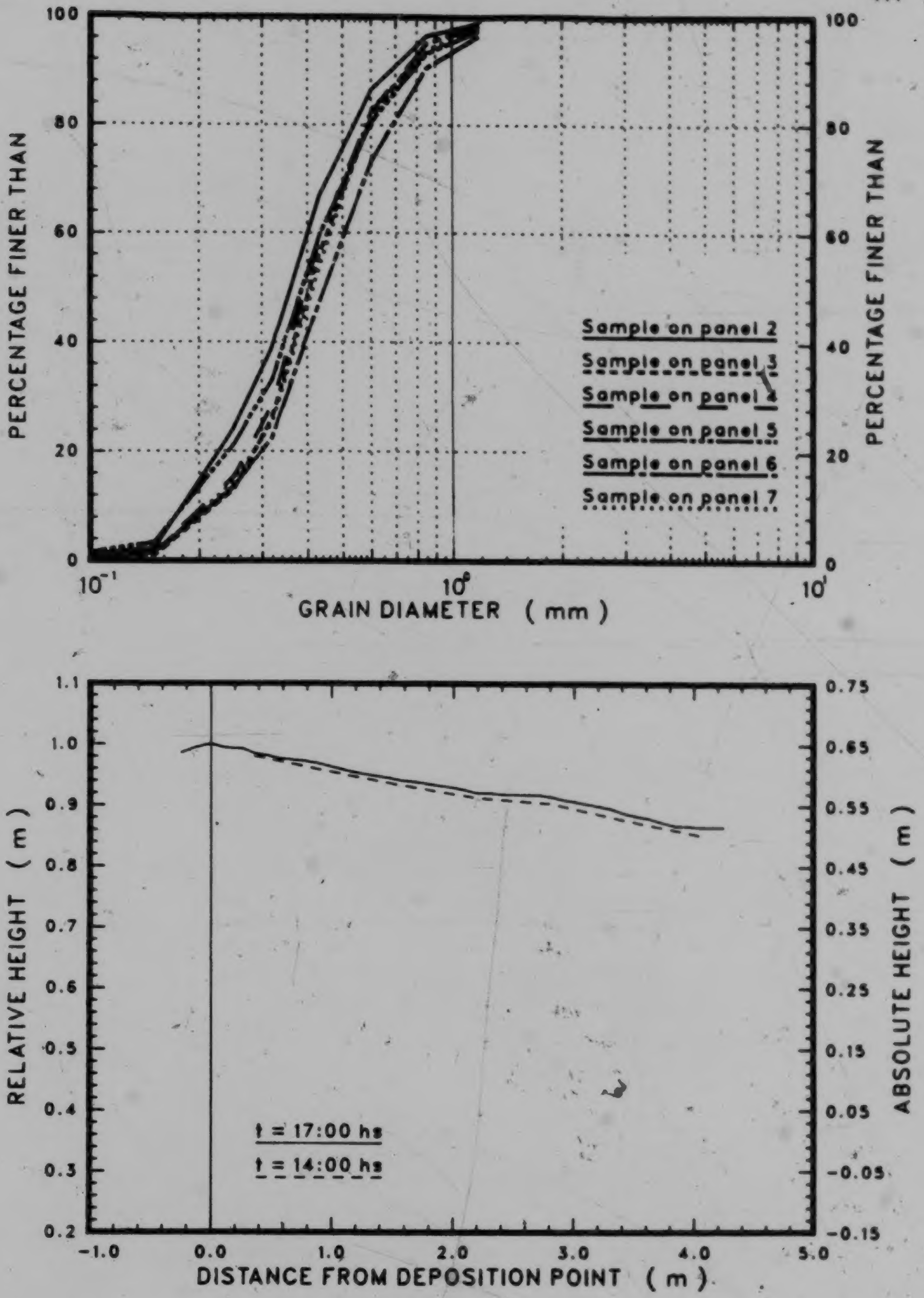


FIG. B.88 – Results of test KS9 (Qt = 10 l/min, Cw = 2 %)

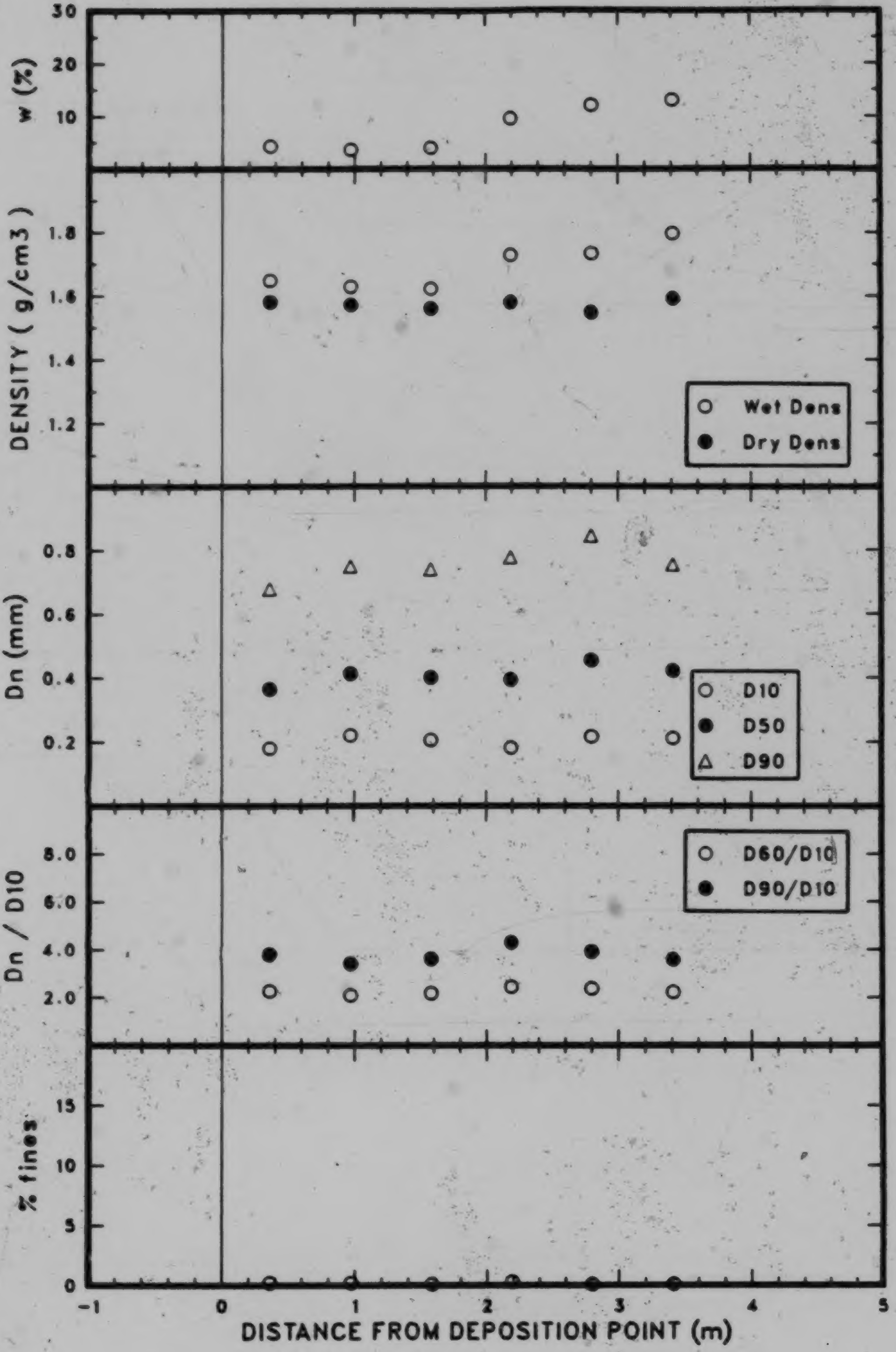


FIG. B.89 – Results of test KS9 (Cont.)

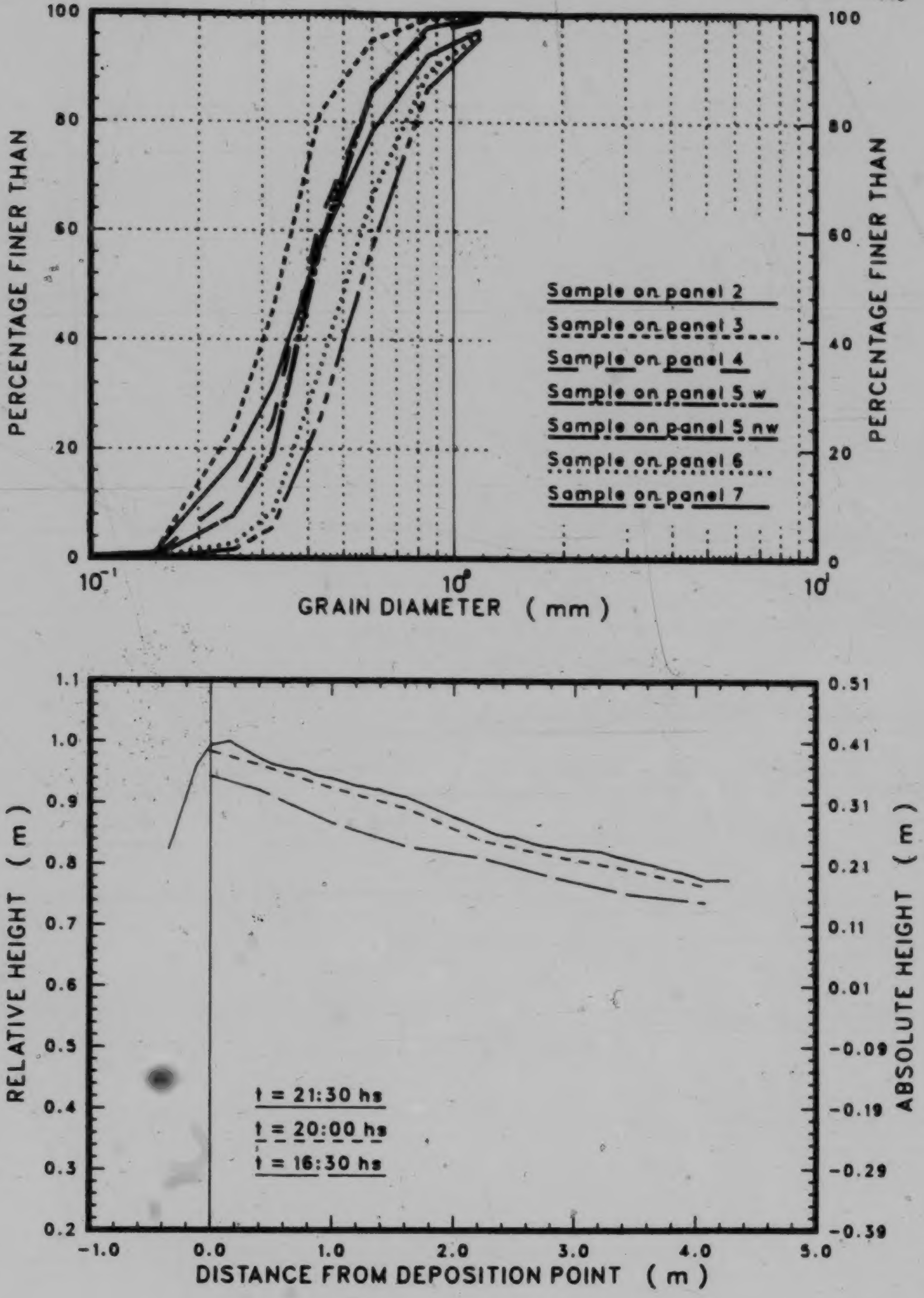


FIG. B.90 – Results of test KS10 (Qt = 10 l/min, Cw = 6 %)

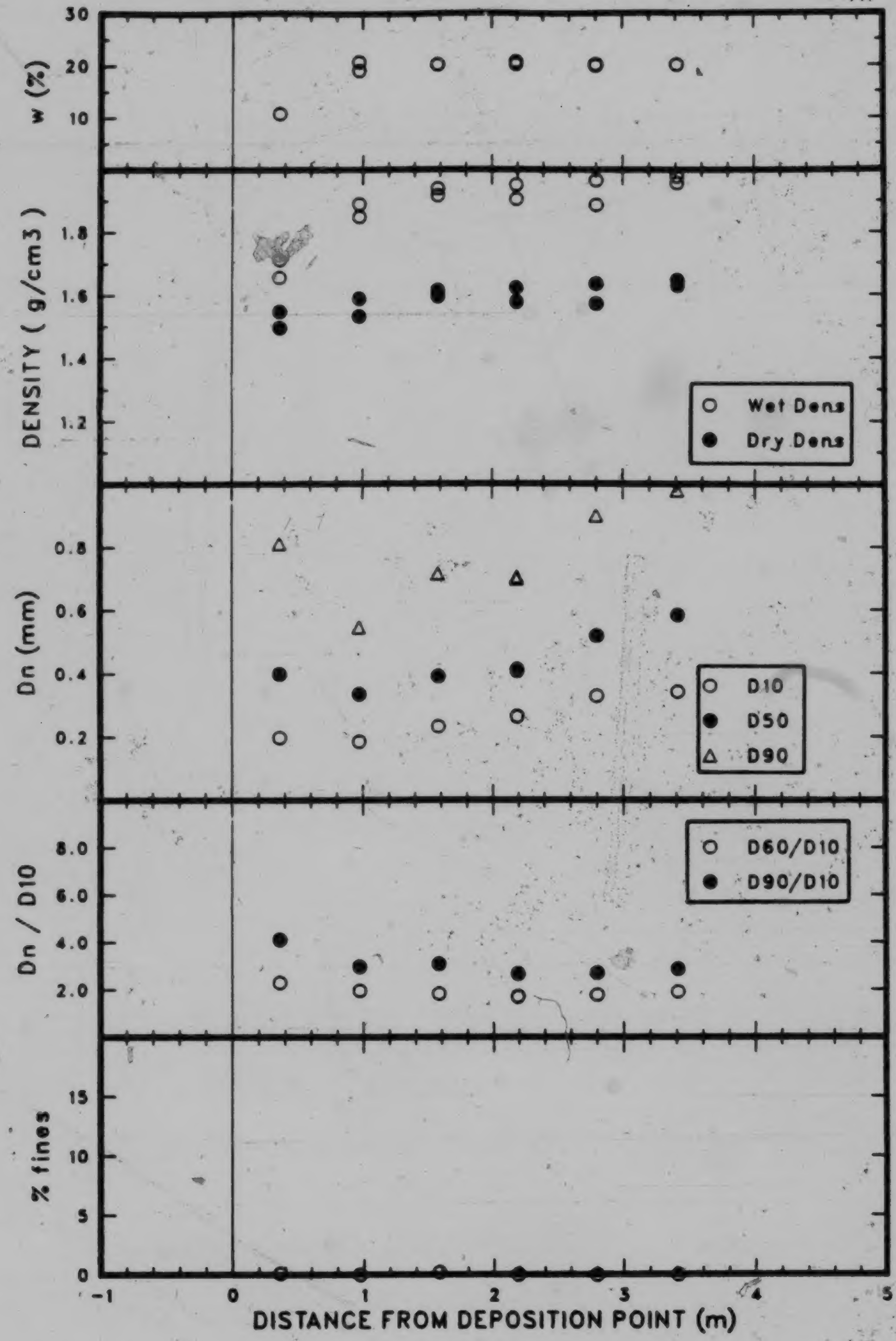


FIG. B.91 – Results of test KS10 (Cont.)

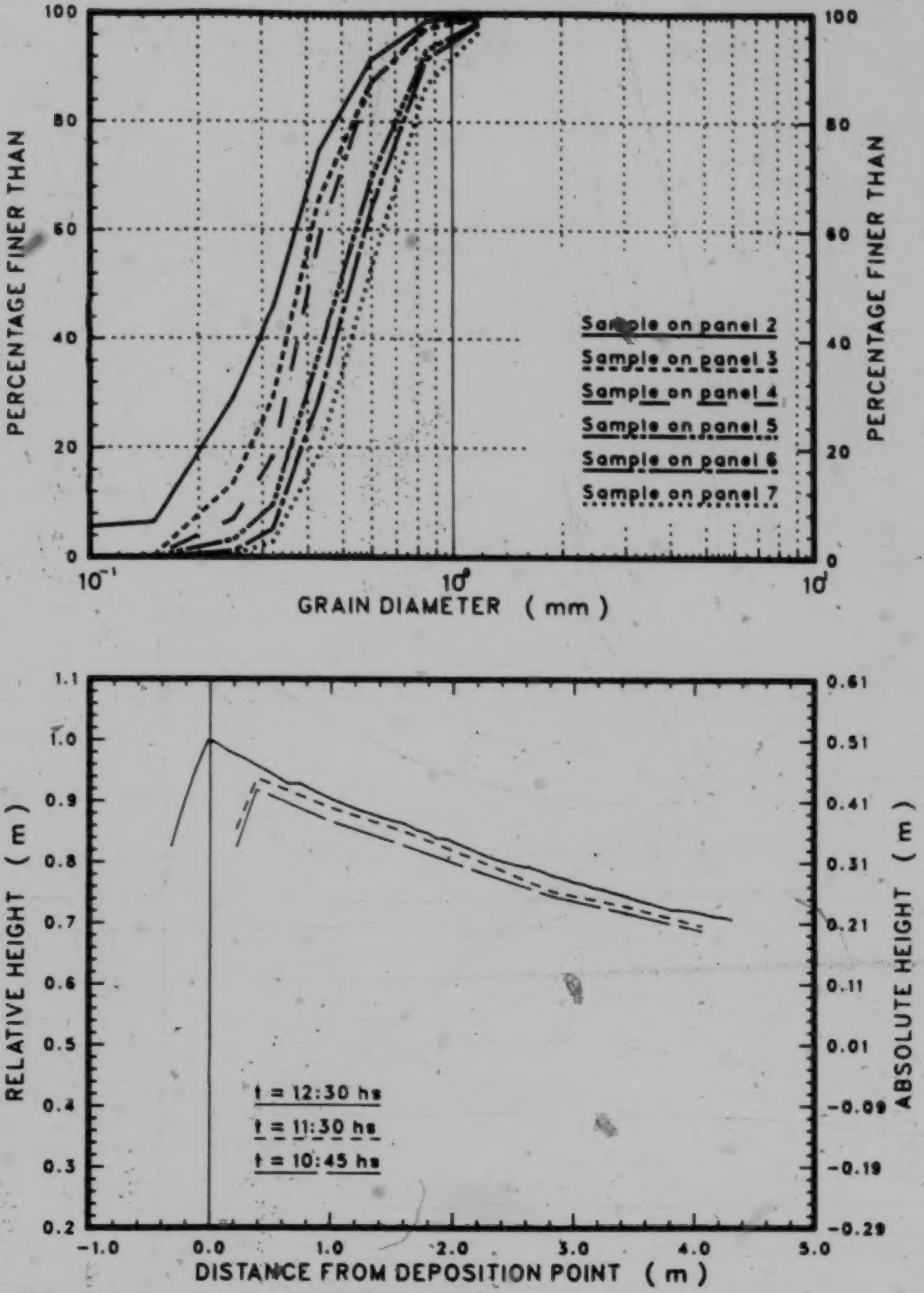


FIG. B.92 – Results of test KS11 (Qt = 10 l/min, Cw = 10 %)

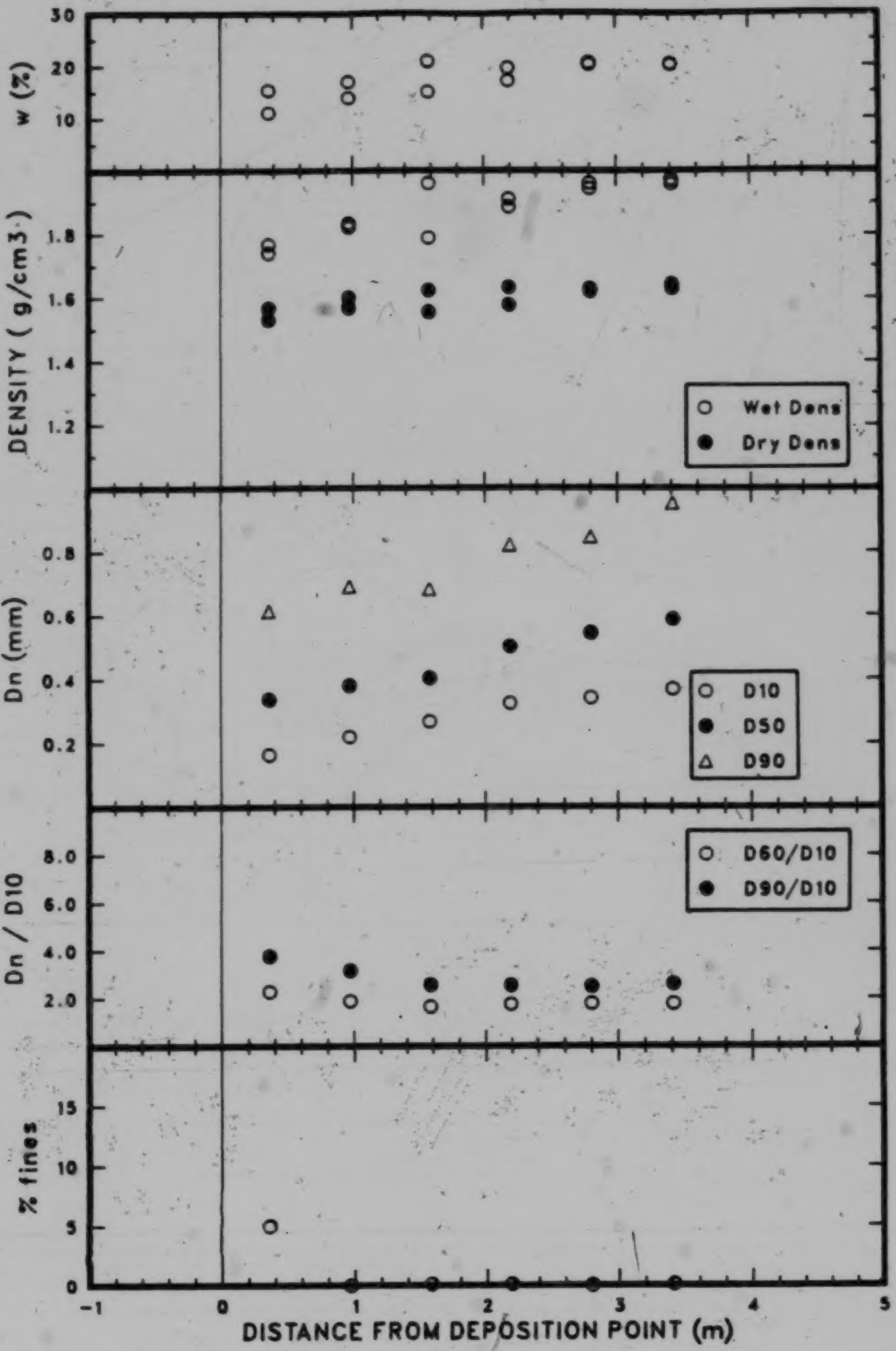


FIG. B.93 – Results of test KS11 (Cont.)

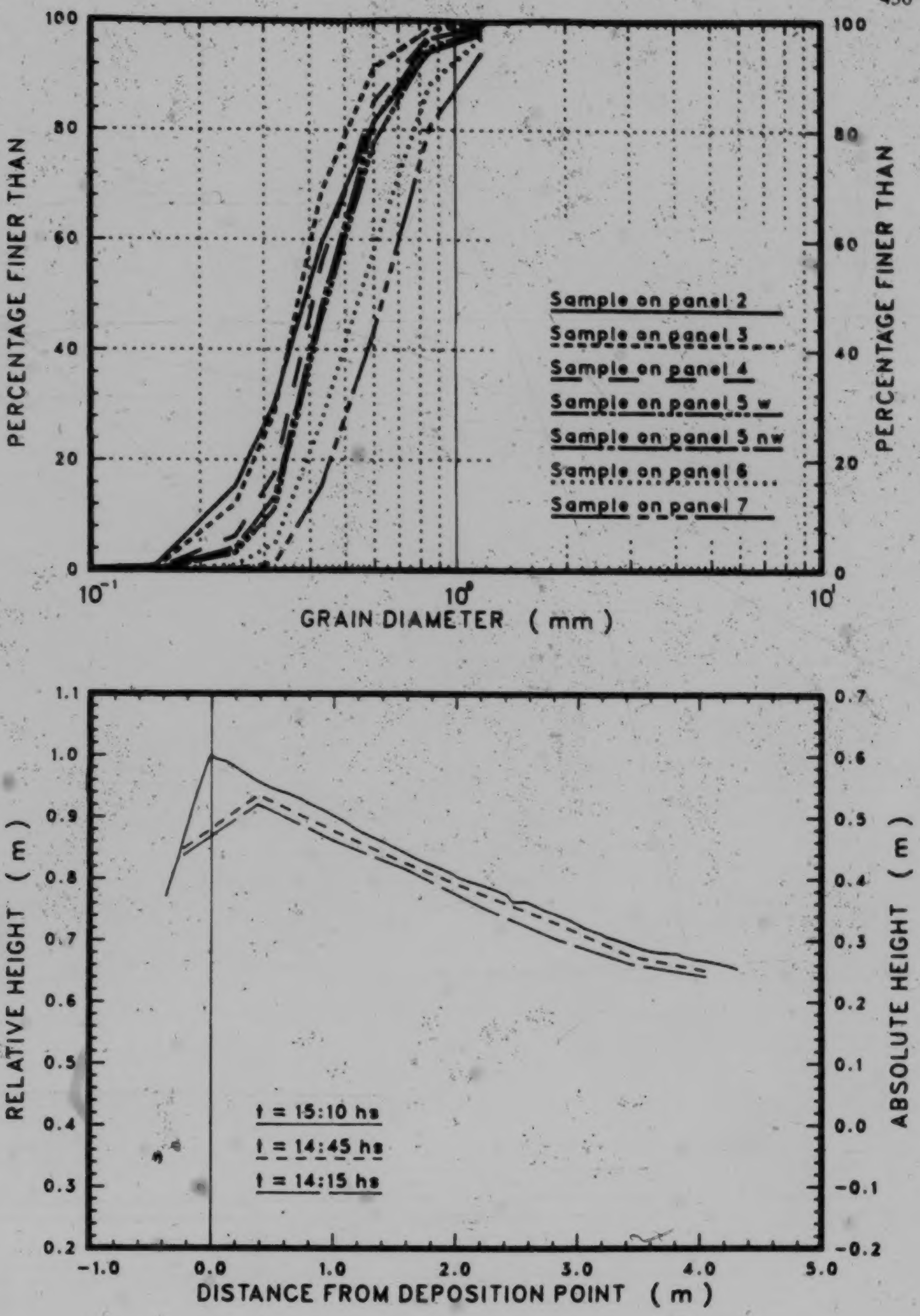


FIG. B.94 – Results of test KS12 (Qt = 10 l/min, Cw = 15 %)

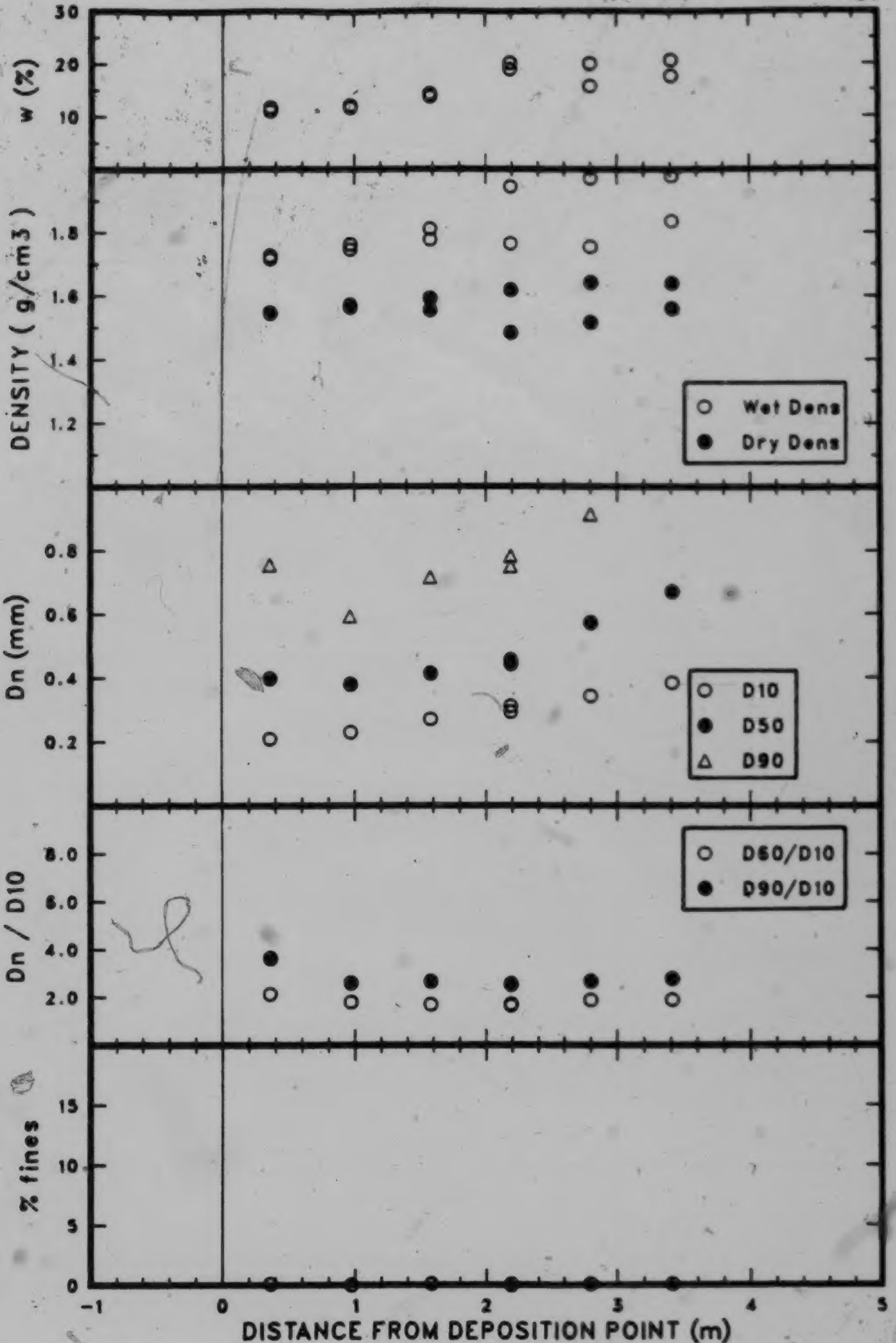
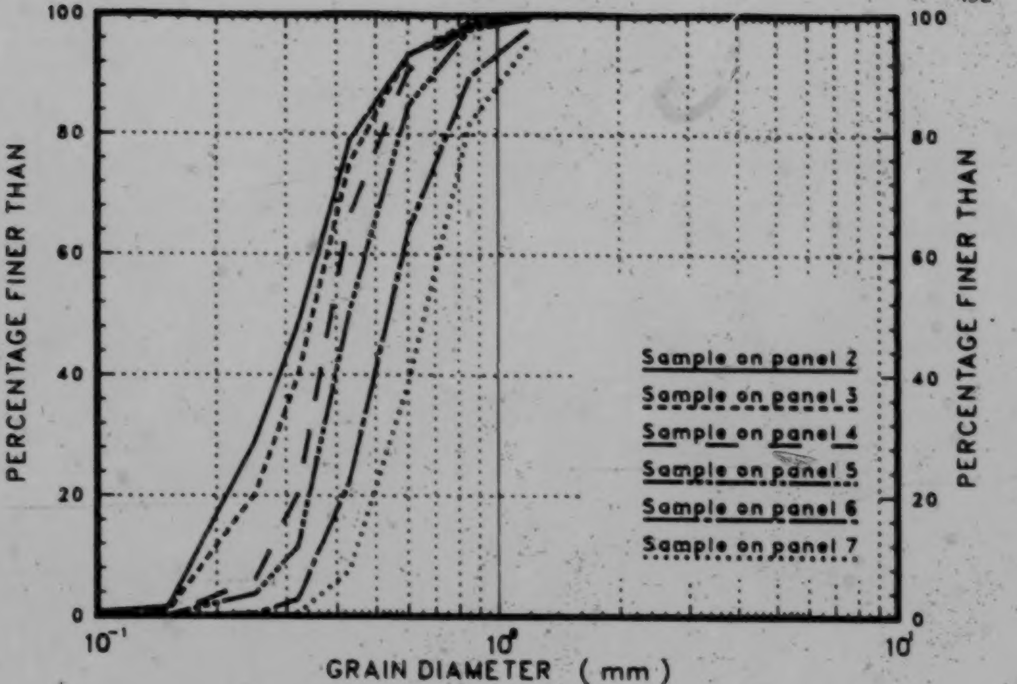


FIG. B.95 – Results of test KS12 (Cont.)



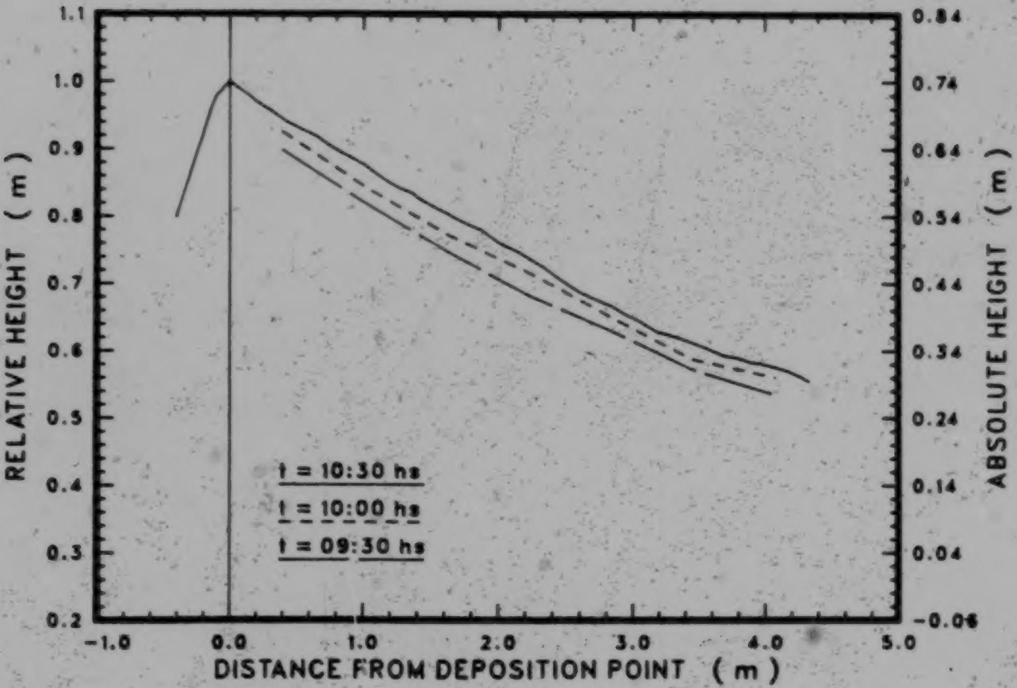


FIG. B.96 – Results of test KS13 (Qt = 10 l/min, Cw = 20 %)

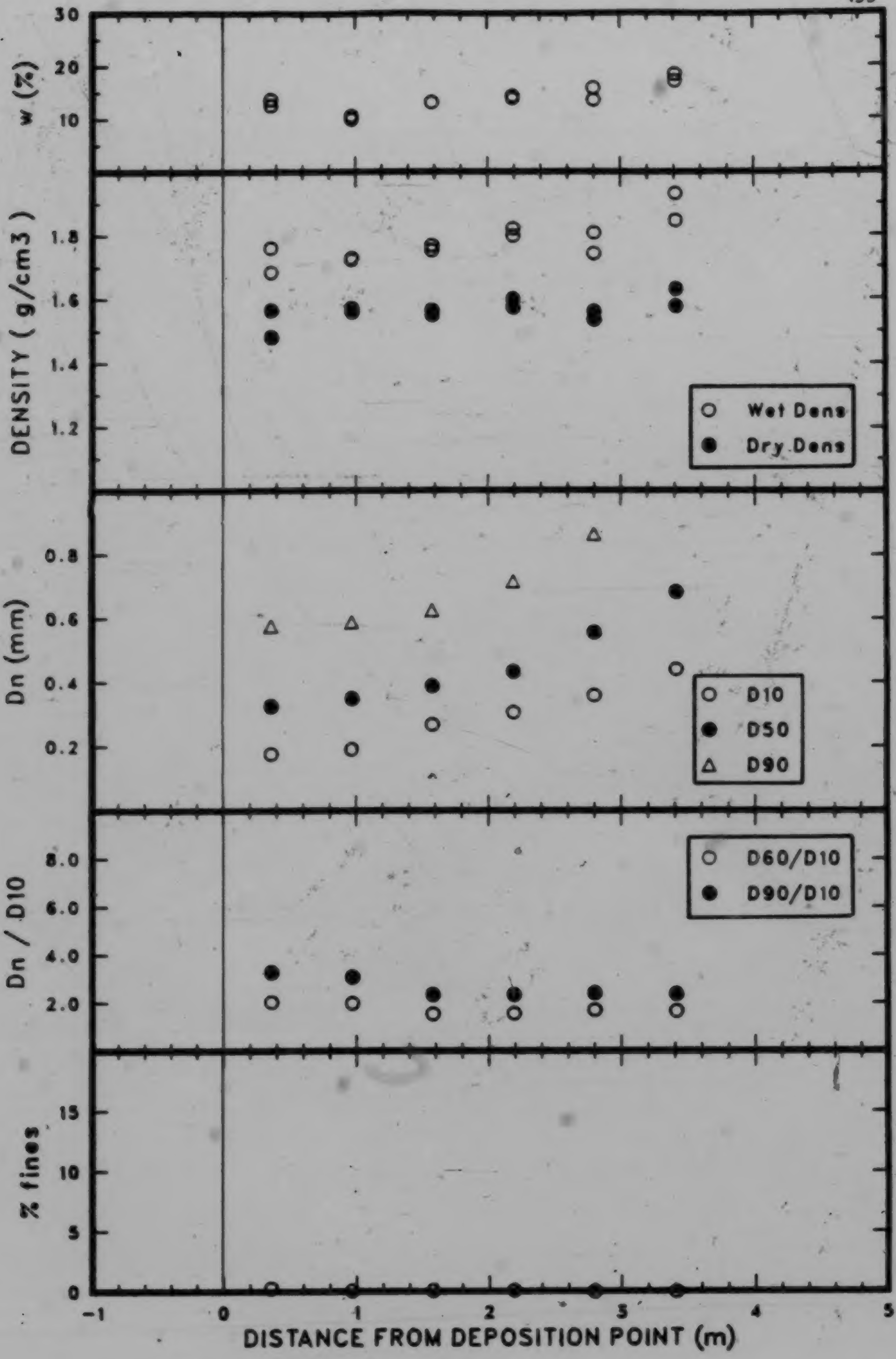


FIG. B.97 – Results of test KS13 (Cont.)

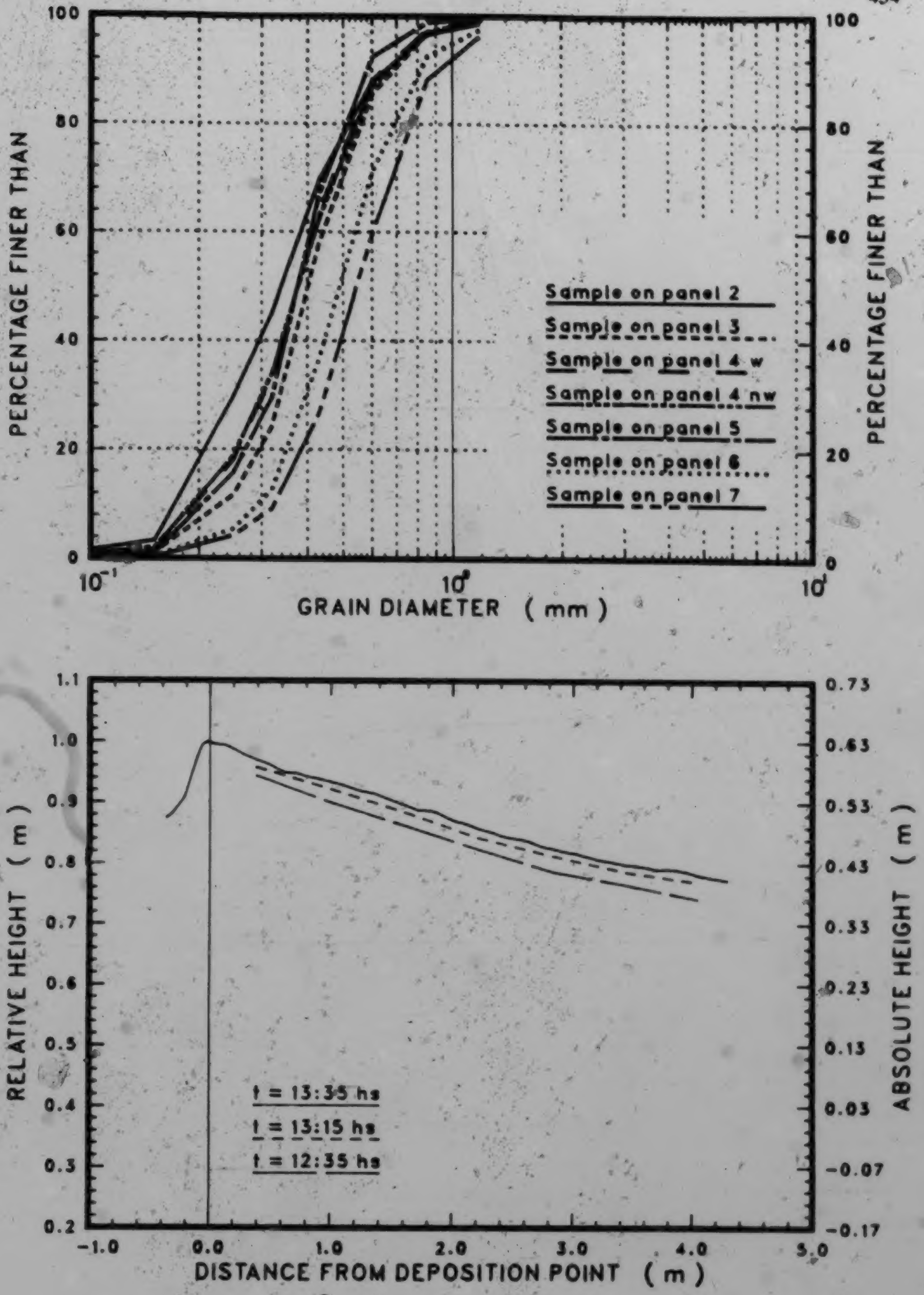


FIG. B.98 – Results of test KS14 (Qt = 20 l/min, Cw = 10 %)

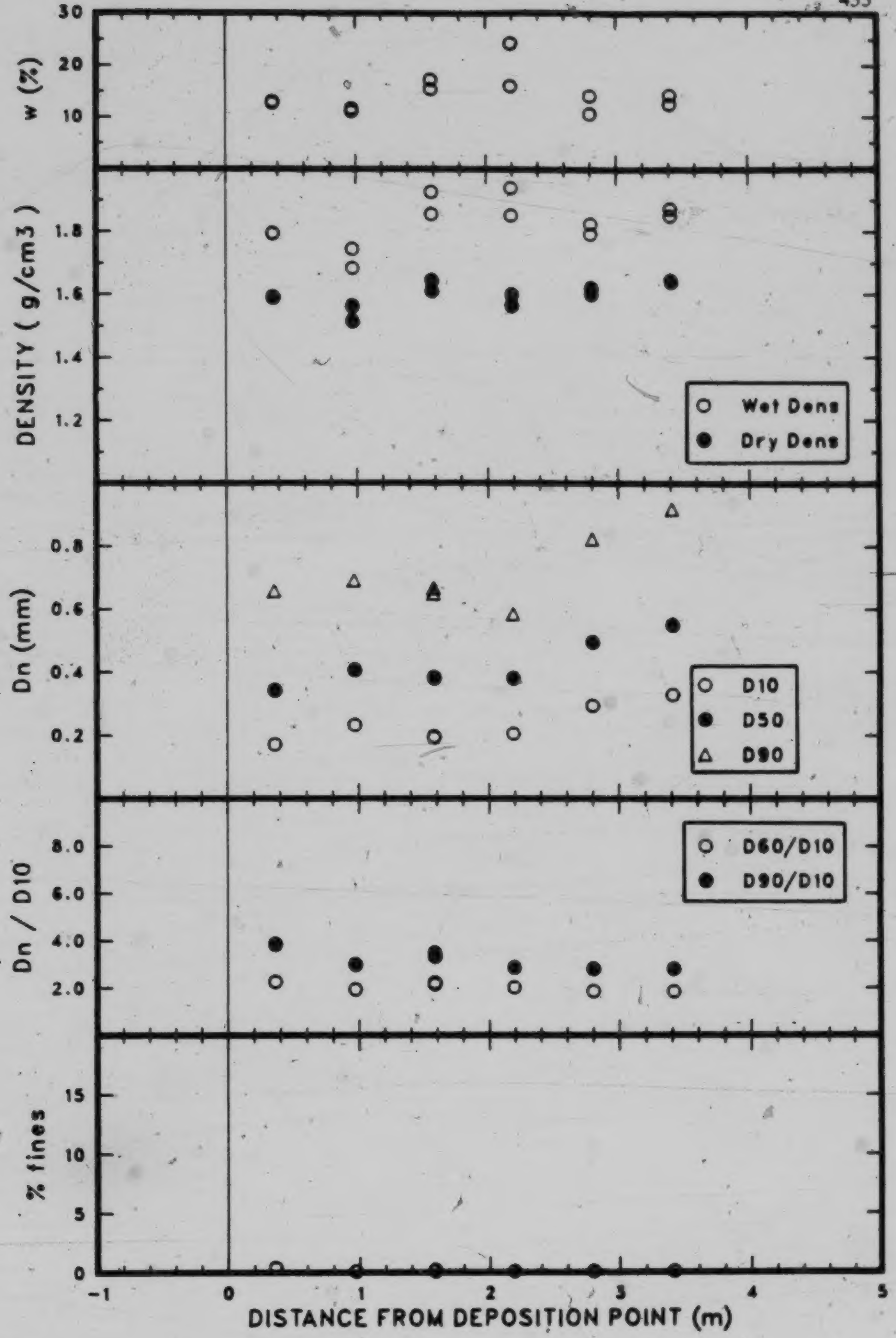


FIG. B.99 – Results of test KS14 (Cont.)

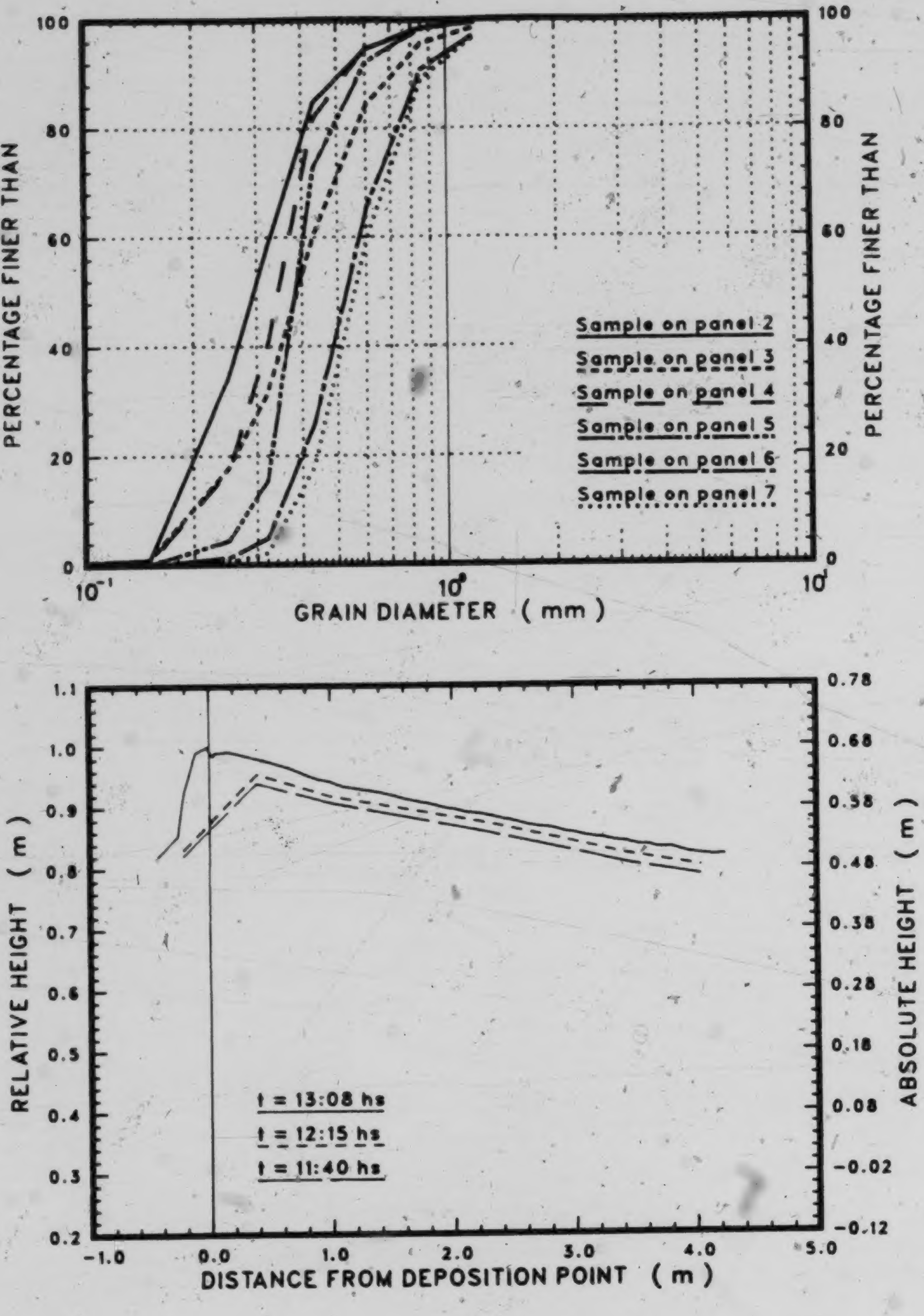


FIG. B.100 – Results of test KS16 (Qt = 20 l/min, Cw = 6 %)

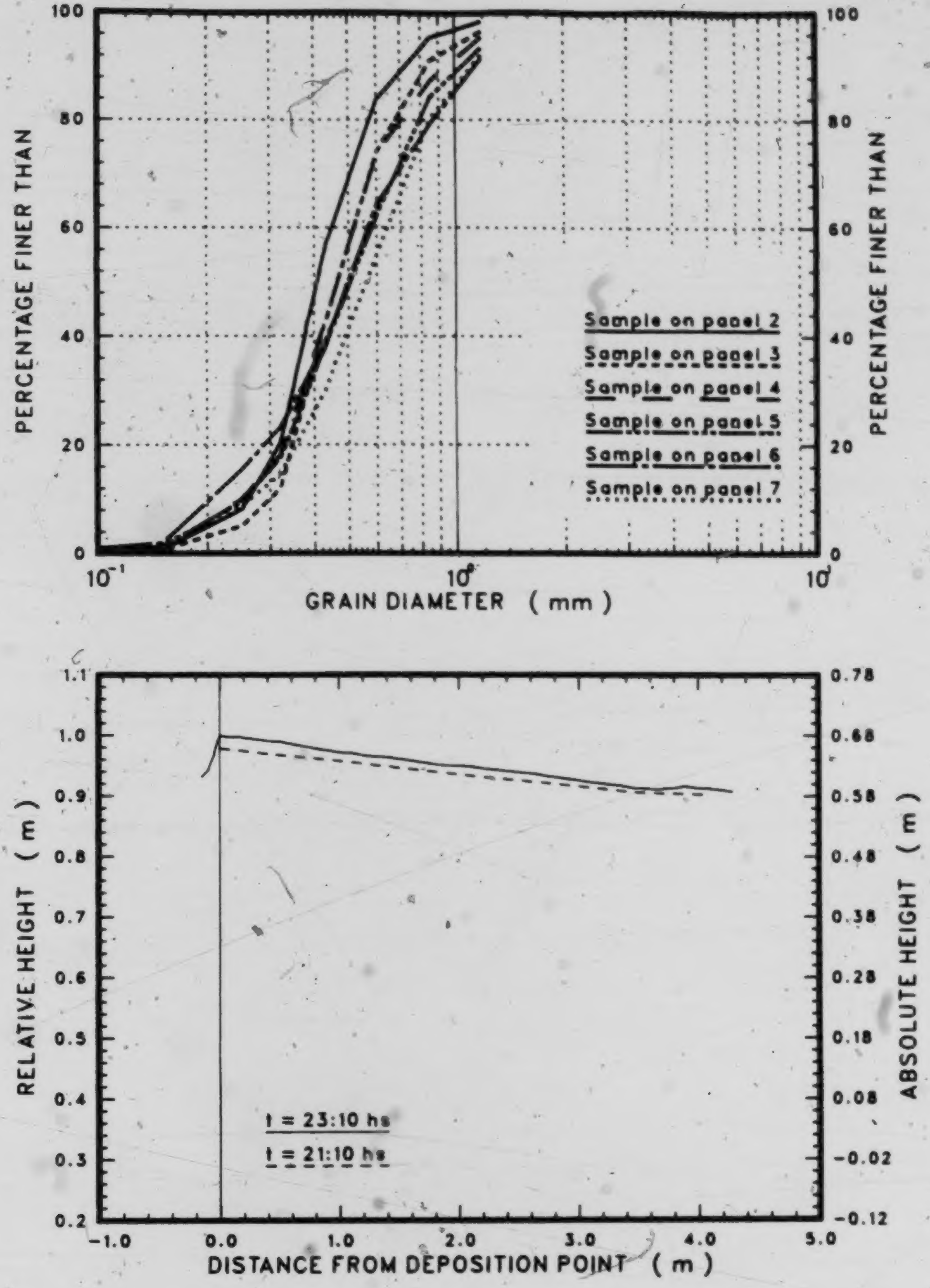


FIG. B.101 – Results of test KS17 (Qt = 20 l/min, Cw = 2 %)

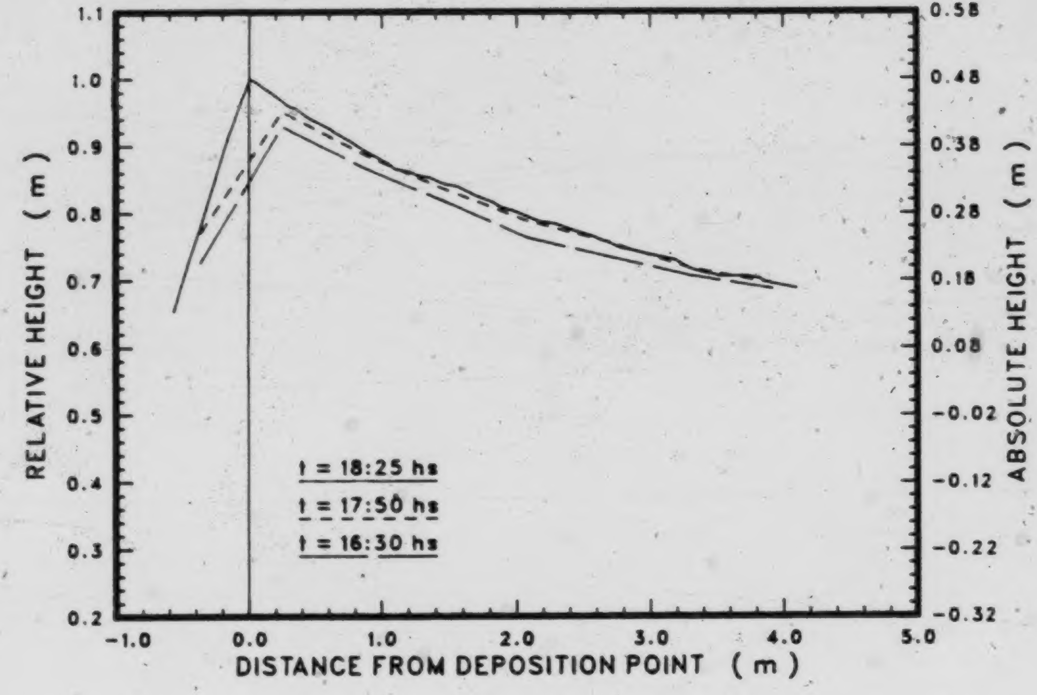


FIG. B.102 – Results of test KS18 (Qt = 5 l/min, Cw = 10 %)

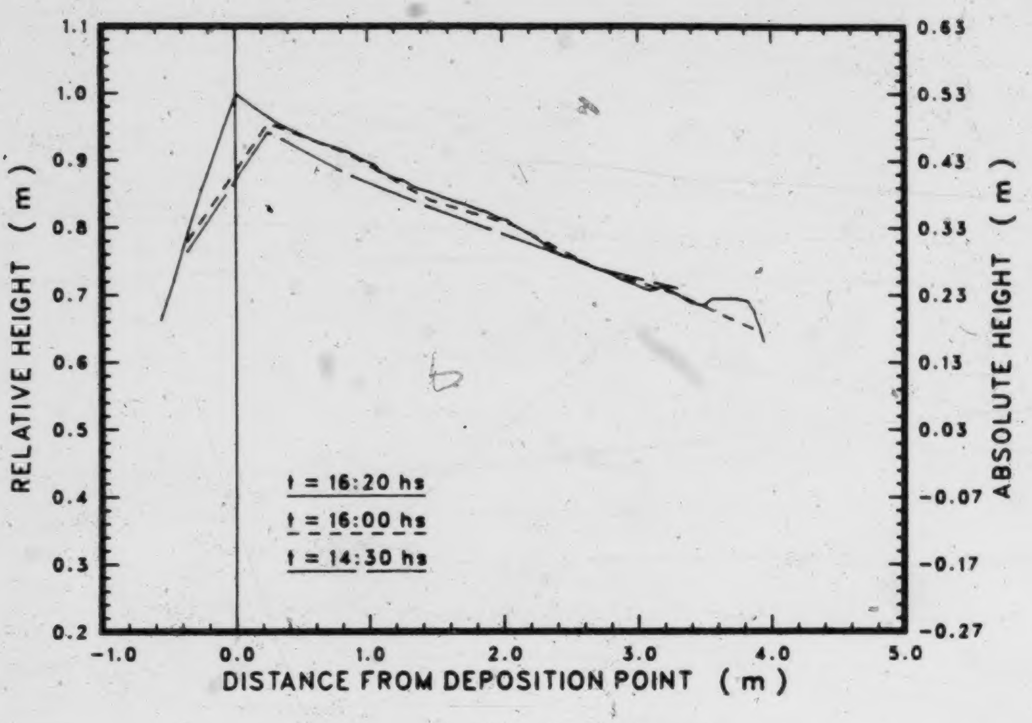


FIG. B.103 – Results of test KS19 (Qt = 5 l/min, Cw = 10 %)

APPENDIX C - DENSITY MEASUREMENTS FOR THE FIELD TESTS

C.1 - INTRODUCTION

The density of the beach material from the various field tests was measured using three different techniques:

a) in-situ measurement with a nuclear probe strata gauge

b) laboratory determination of density of frozen undisturbed samples using the wax method

c) laboratory determination of density of frozen undisturbed samples by direct determination

The results obtained using each of the three methods are summarized and compared in this appendix.

C.2 - NUCLEAR STRATA GAUGE

The nuclear strata gauge used to measure densities in-situ was a double probe gauge developed by Campbell Pacific Nuclear Corp. for agricultural applications, fitted with the electronics from the model 501DR gauge. The 60 cm long probes can measure moisture content and density every 5 cm. The right hand probe contains a radioactive source at the bottom of the rod. The source is a combination unit with 10 mCi of Cs-137 as a gamma source for density measurements and 50 mCi of Am-241/Be as a source of fast neutrons for moisture content measurements. Immediately above the source there is a He3 (thermal) slow neutron detector. The left hand probe contains a GM detector. The gamma particles passing between the two probes and detected on the left side provide an inverse measurement of the soil density between the probes. The neutrons thermalized in the vicinity of the right hand probe constitute a direct measurement of hydrogen in the soil, which can be correlated to the moisture content.

The strata gauge was calibrated in the laboratory using material brought from the field, as the calibration equations depend on the soil type. The calibration was made by placing the soil in a large box (80 cm x 80 cm x 100 cm) at known moisture content and density, lowering the probes of the strata gauge placed on the surface of the soil and obtaining the readings in both moisture and density channels using the same procedure as in the field. The material was placed in the box in thin layers, and several samples were collected from each layer for moisture content determination. After placing each layer, the height of the material in the box was measured in 12 different locations and the weight was recorded (the box was laid on a scale), in order to control the value of the soil density. Calibration equations were obtained for both moisture and density channels. Each channel requires two or three separate equations as the most superficial readings should be treated separately.

The values of density and moisture obtained in the field for several sites in each test area are presented in Figures C.1 to C.60. These graphs also show the density of undisturbed samples taken at the same point, as measured by the wax method. The code of each location (e.g. SG34) is the same used for the undisturbed samples as strata gauge measurements and sampling were, in most cases, carried out at the same location or only a few decimeters apart.

The strata gauge has the advantage of providing in-situ measurements of both density and moistures content at several depths. The close spacing between the readings (5 cm) and the limited maximum depth (60 cm) reflects the fact that the equipment was developed for agricultural uses. It was adequate for this research since the objective was to study the effect of the deposition mode on the beach; the deposits were not very thick and effects of depth were not being considered. The operation of the strata gauge proved to be time consuming compared to sampling. A minor difficulty with the weight of this equipment was solved by using a sled to move it from one location to the other. Problems were encountered during operation at low temperatures under winter conditions.

C.3 - UNDISTURBED SAMPLES

Undisturbed samples were obtained either by drilling the frozen beach with a 4 inch hand-barrel or by statically pushing down 4-inch sampling tubes, excavating around them and freezing the material from the bottom ~~up~~ with carbon dioxide pellets (dry ice) in a controlled manner. Thin walled PVC samplers produced more uniform freezing with less heave of the samples than metal samplers.

The undisturbed samples were kept frozen in large insulated boxes containing dry ice and were sent to the laboratory at the University of Alberta in Edmonton, where they were trimmed in a -25° C walk-in freezer to remove the outer parts considered disturbed (see Küpper, 1991, Chapter 3). The samples were kept frozen until required for testing.

Regular shaped pieces of the frozen samples were cut for determination of density both by the wax method and by direct measurement, which was performed by measuring several times the dimensions of the specimen with a caliper and by obtaining its weight, frozen and dry. The accuracy of this method depends on the regularity of the specimen and on the presence of small indentations, which were common on samples that had pronounced.layering. The determination of density by the wax method followed the ASTM procedure. A small gap of air or vapour was observed to form between the wax and the samples, affecting the accuracy of the measurements. The results of density determination in undisturbed samples using both methods are presented in Figures C.61 to C.64, which also compare these results to the obtained using the strata gauge.

C.4 - COMPARISON OF RESULTS

Figures C.61 to C.64 show the dry density values measured using the wax method (horizontal axis) plotted against the values of dry density obtained by direct measurement and by in-situ strata gauge determinations. For Test 0 (Figure C.61), direct measurement resulted in density values lower than obtained with the wax method, while strata gauge results were higher. For Test 1 both laboratory methods yielded comparable results, although the scatter is significant (Figure C.62). In only one location in the Test 1 area strata gauge and sampling were performed at the same location and the results are not very close as shown in Figure C.62. Densities obtained by the wax method and by direct measurement were similar for samples from Tests 2 and 4 (Figures C.63 and C.64). However, strata gauge densities were higher than densities measured in undisturbed samples for Test 4 and significantly lower for Test 2.

C.5 - CONCLUSIONS AND RECOMMENDATIONS

Density determination by the wax method and direct measurement of density yielded consistent results for frozen undisturbed samples. A significant scatter was observed, however the measurements were performed in different pieces of the same sample, what explains, at least in part, such a variability. The hydraulic deposition process results in the formation of thin layers that can differ slightly from each other in composition and structure, and therefore in density. Some undisturbed samples of Test 2 (SG5, SG9, SG10, SG11 and SG22) were sliced and each piece had its density measured using the wax method; a significant difference in density was observed from slice to slice (see Table 5.9, Chapter 5).

Nuclear strata gauge results differed from both laboratory methods to a larger extent and the discrepancy did not present a consistent trend that could have been assimilated into the calibration equations. Consequently, strata gauge results were not considered in the analysis of the tests results.

The values of density determined by the wax method were considered the most reliable ones and were the only values included in the analysis of the field results discussed in this thesis. The wax method has been traditionally adopted in geotechnical engineering and its procedure and limitations are well known. Recommendations for future work using the wax method to determine the density of frozen samples include the use of a wax with a low melting point and the largest possible sample size in order to minimize the error introduced by the gap caused by vapour between the sample and the wax layer. A large sample of a hydraulic fill would also include several layers and be more representative of the deposit.

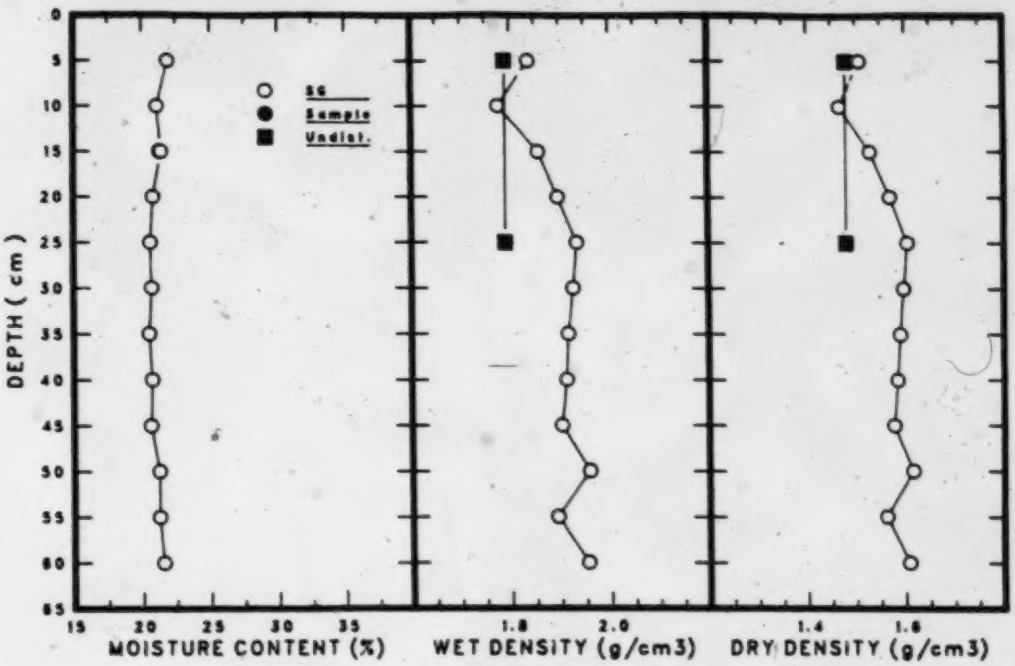


FIG. C.1 – Strata gauge measurements at SG 1 – TEST 0

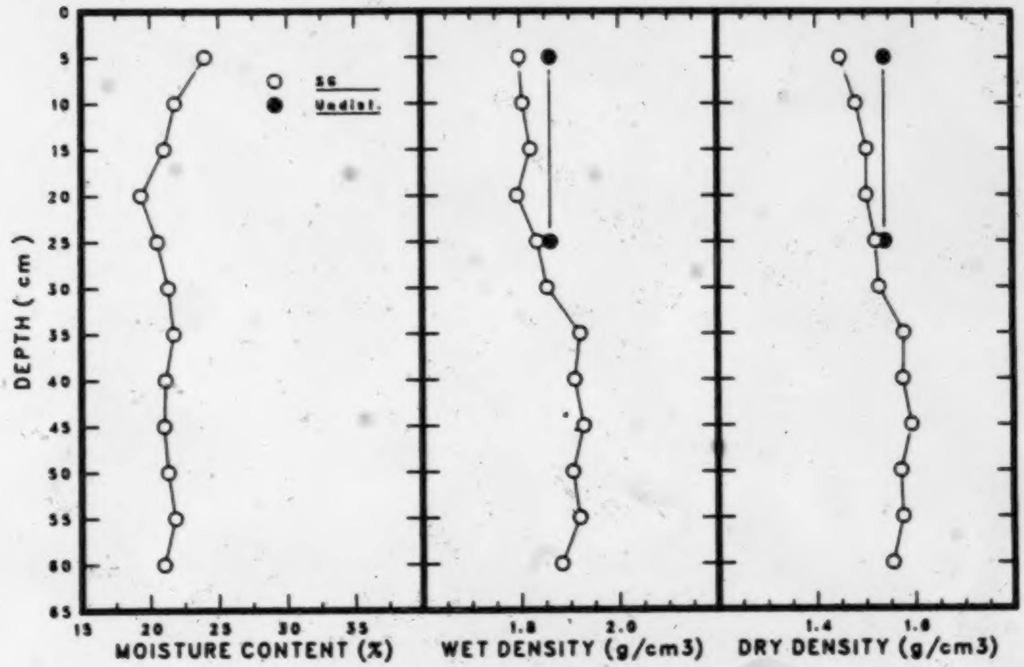


FIG. C.2 – Strata gauge measurements at SG 2 – TEST 0

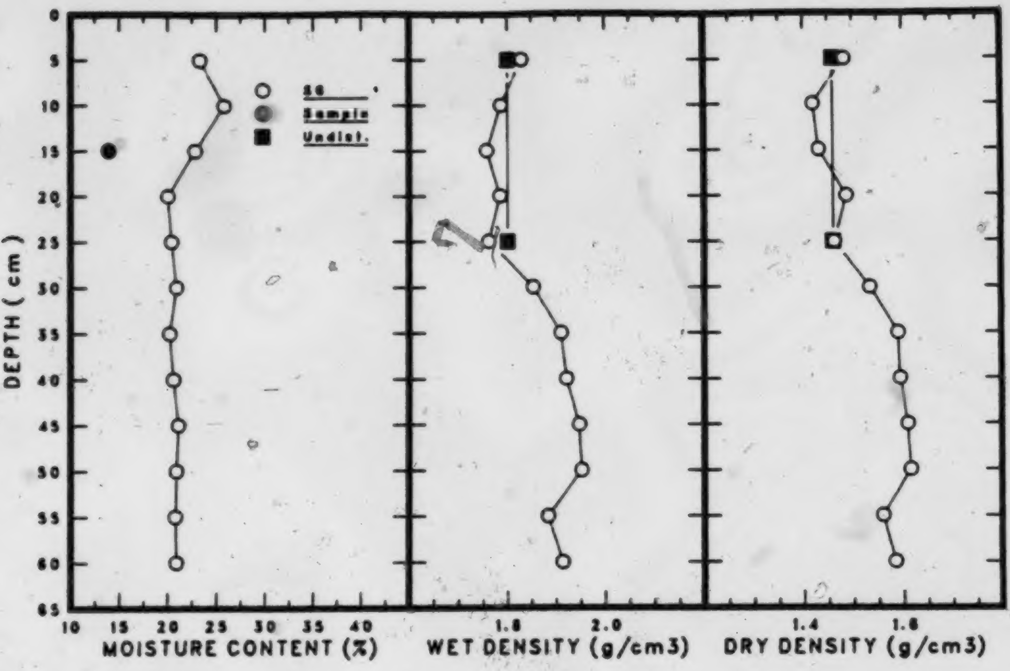


FIG. C.3 – Strata gauge measurements at SG 3 – TEST 0

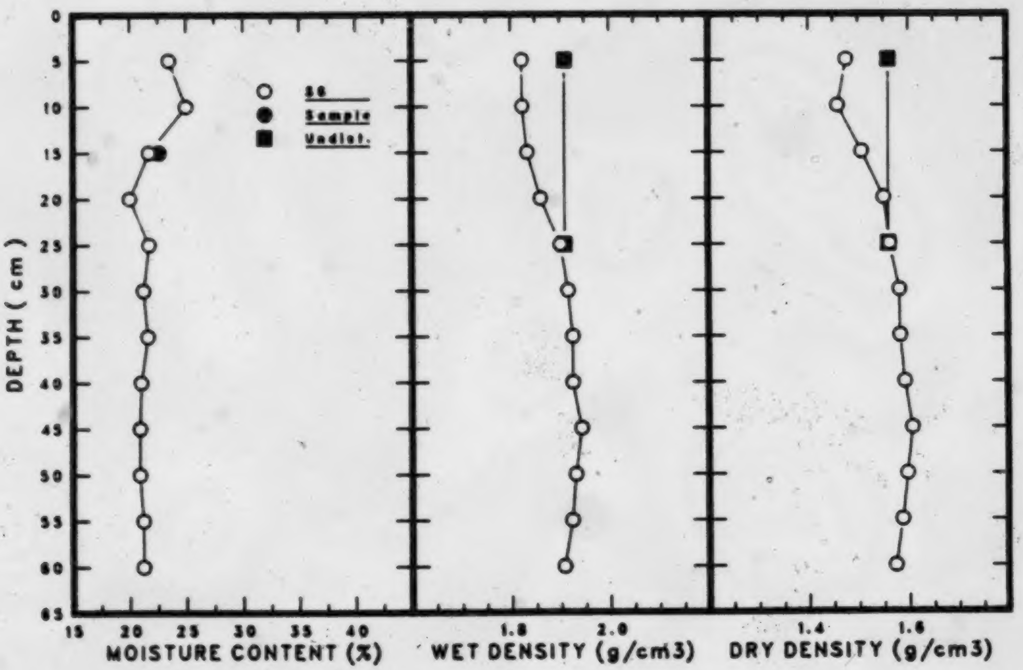


FIG. C.4 – Strata gauge measurements at SG 4 – TEST 0

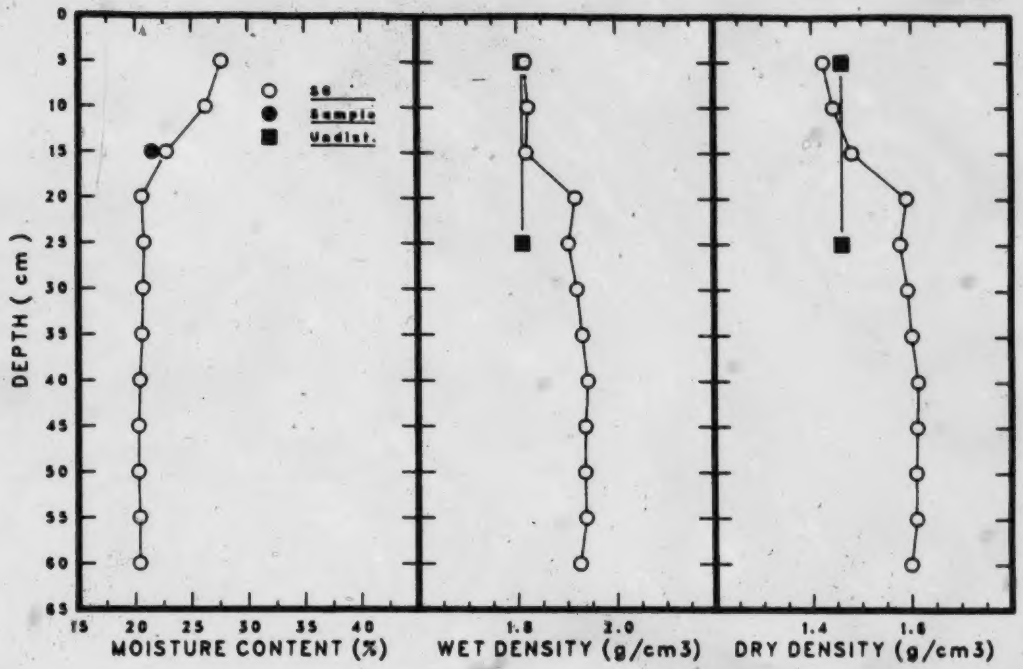


FIG. C.5 – Strata gauge measurements at SG 5 – TEST 0

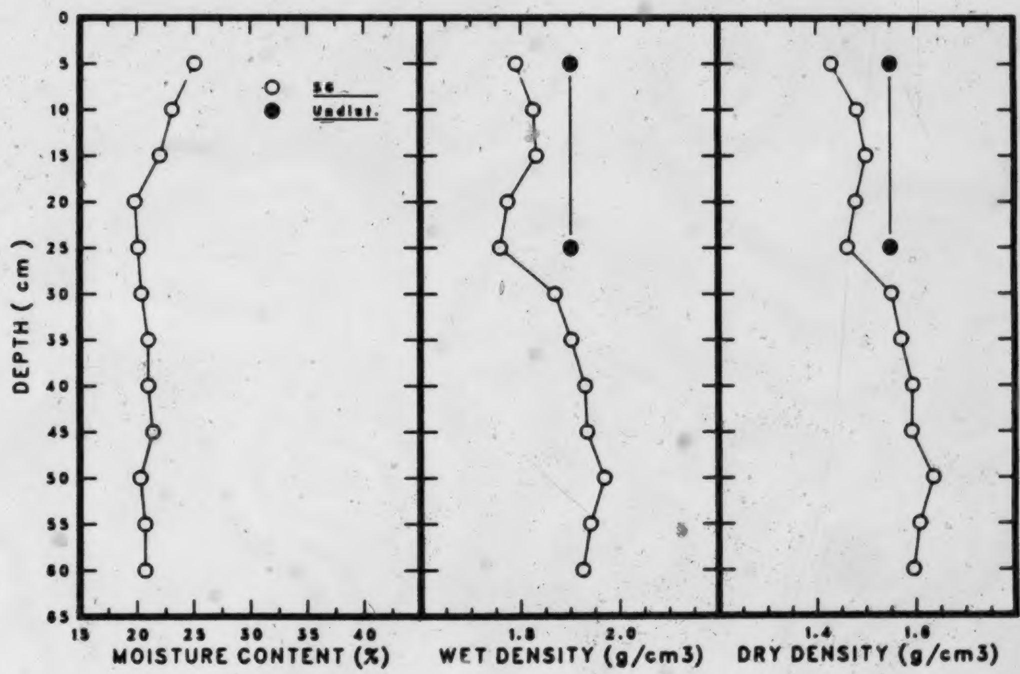


FIG. C.6 – Strata gauge measurements at SG 6/7 – TEST 0

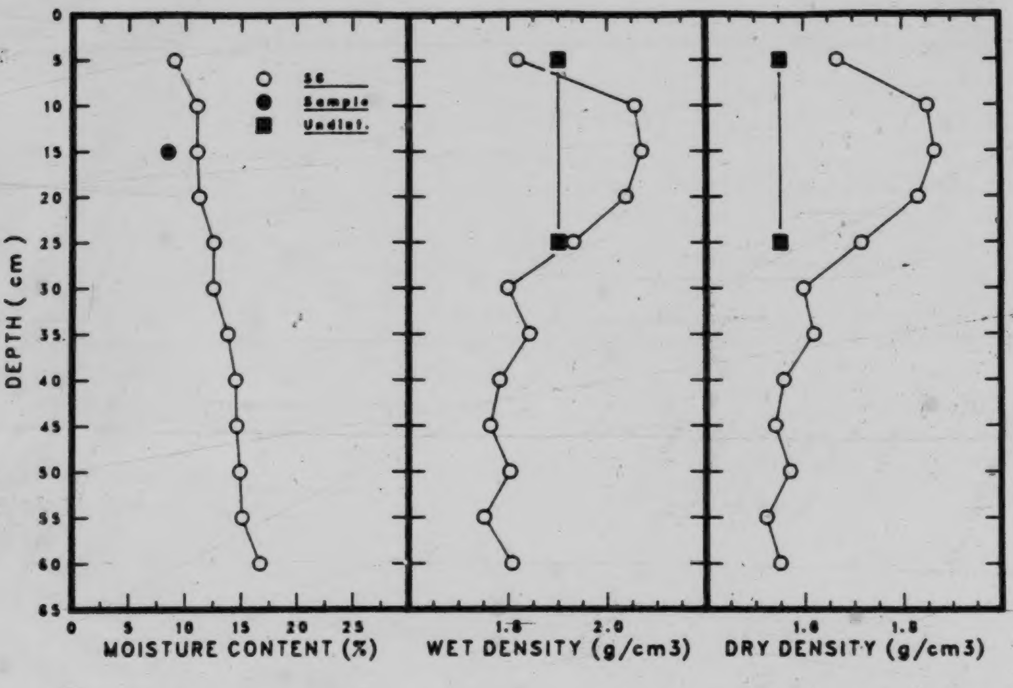


FIG. C.7 – Strata gauge measurements at SG 6B – TEST 0

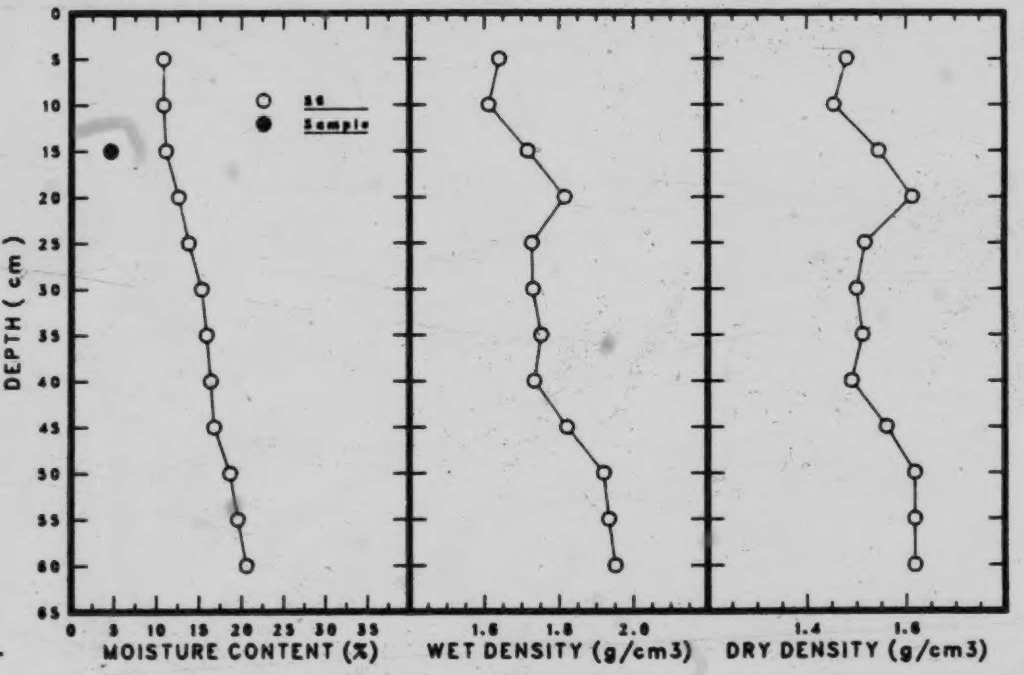


FIG. C.8 – Strata gauge measurements at SG 8 – TEST 0

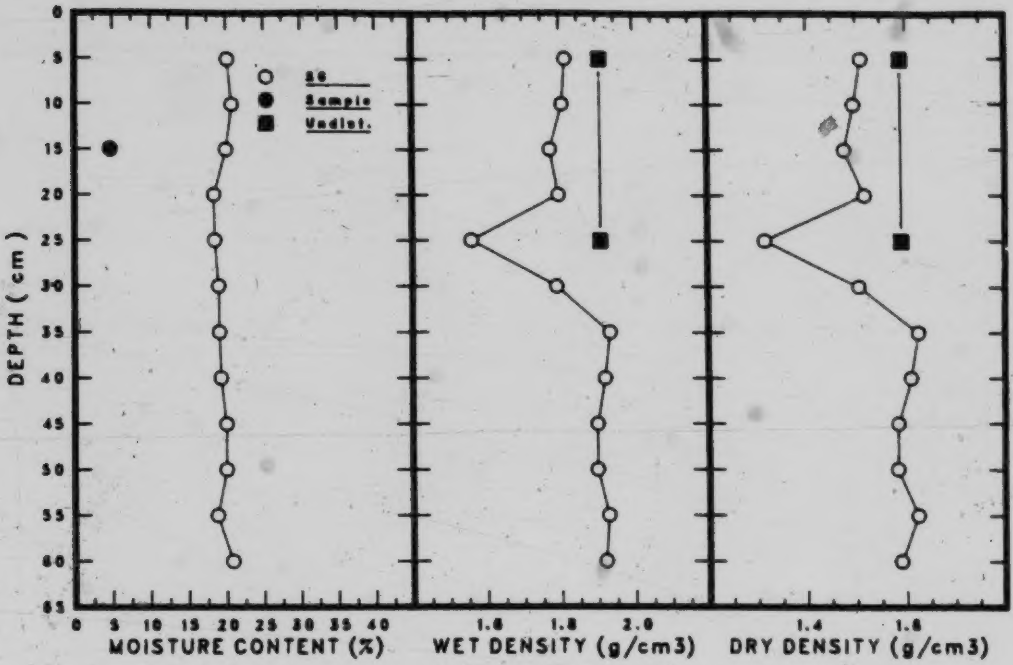


FIG. C.9 – Strata gauge measurements at SG 11 – TEST 0

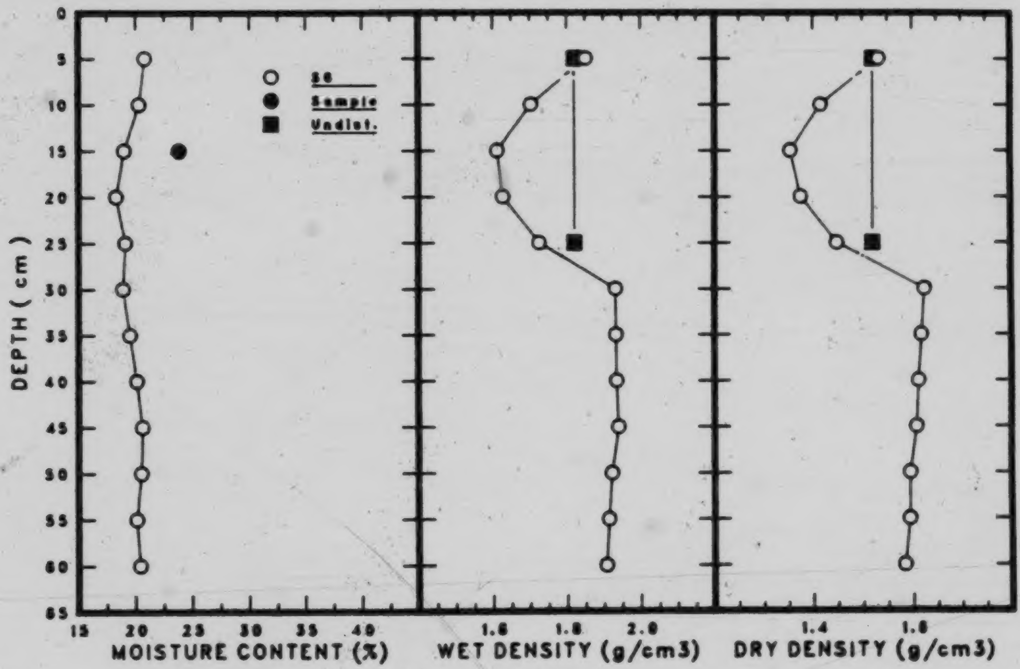


FIG. C.10 – Strata gauge measurements at SG 12 – TEST 0

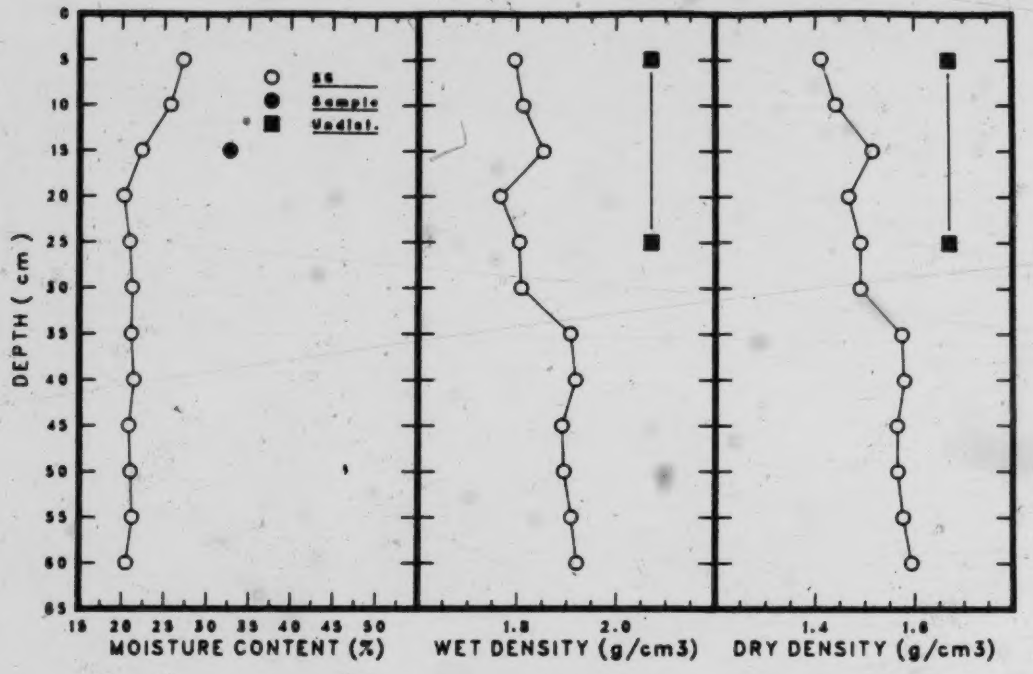


FIG. C.11 – Strata gauge measurements at SG 13 – TEST 0

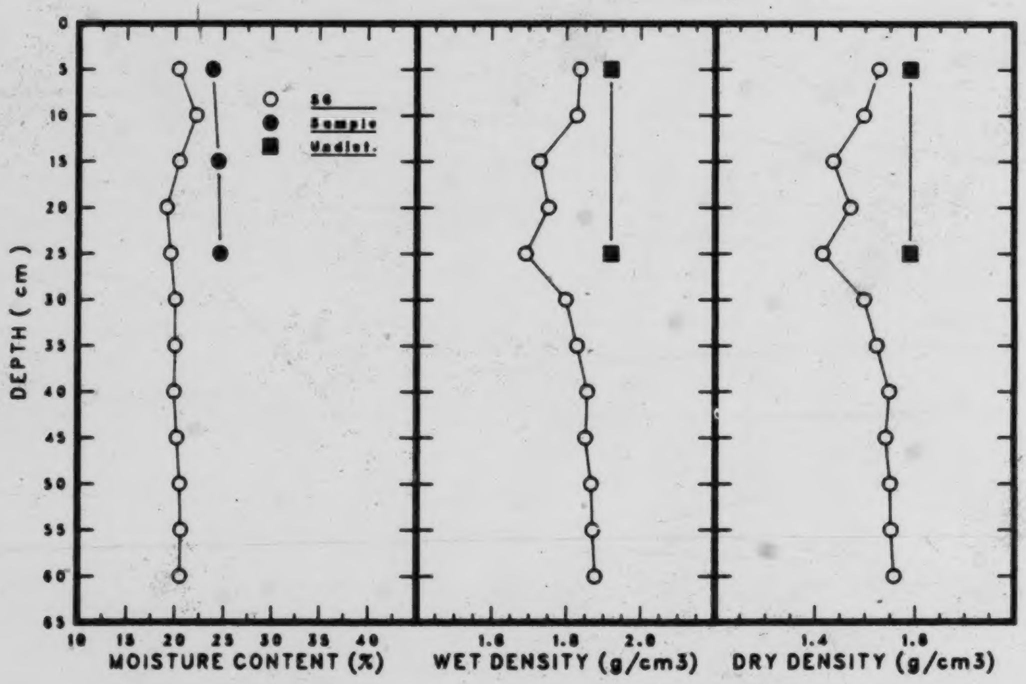


FIG. C.12 – Strata gauge measurements at SG 14 – TEST 0

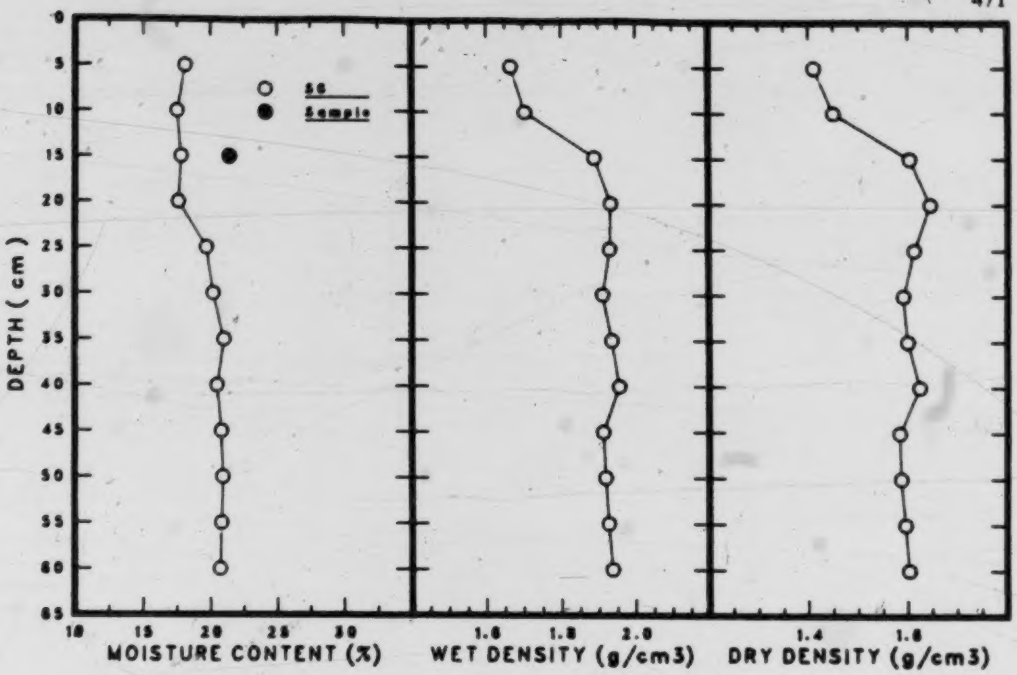


FIG. C.13 – Strata gauge measurements at SG 16 – TEST 0

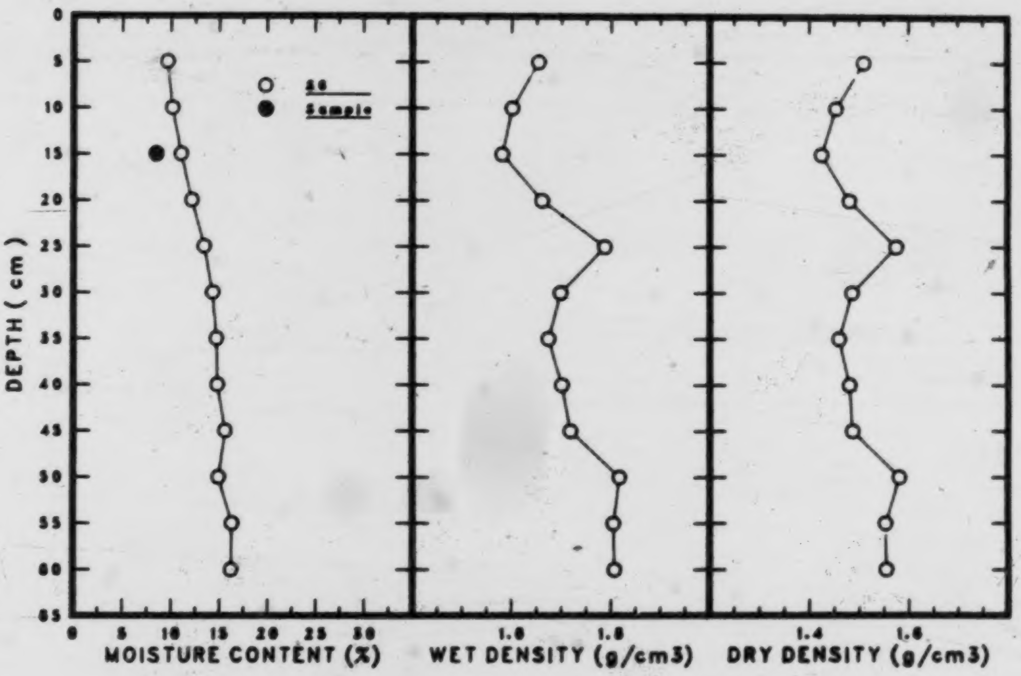


FIG. C.14 – Strata gauge measurements at SG 18 – TEST 0

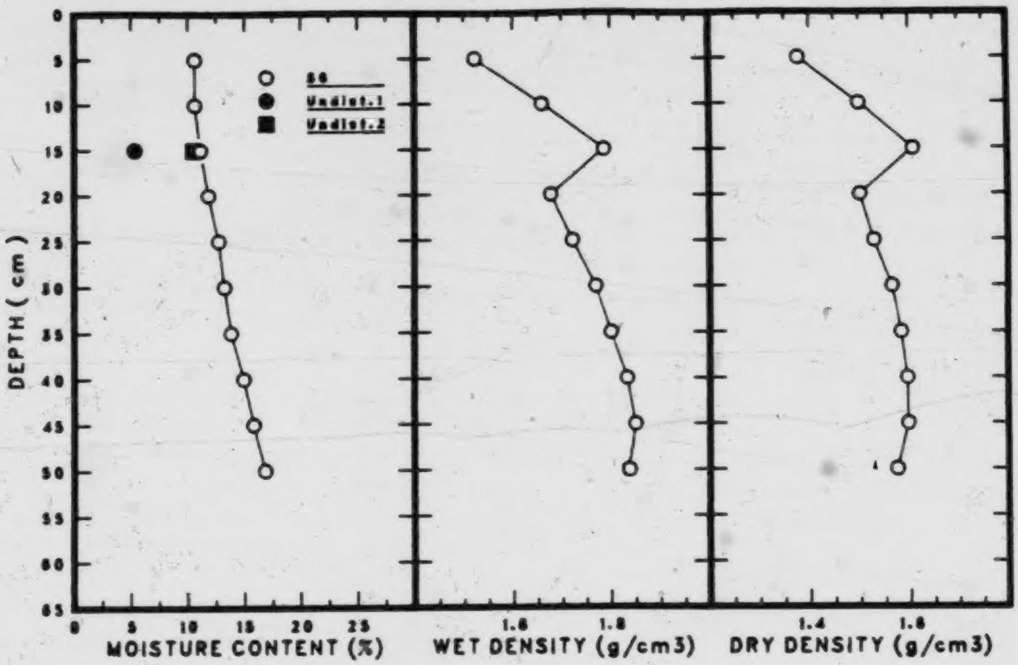


FIG. C.15 – Strata gauge measurements at SG 20 – TEST 0

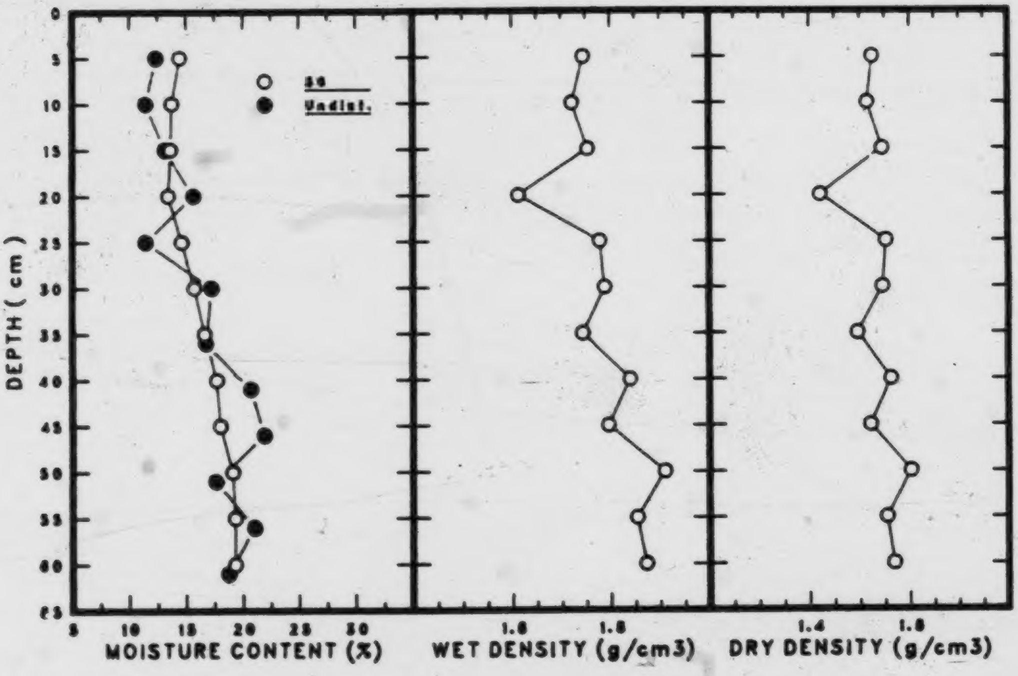


FIG. C.16 – Strata gauge measurements at SG 21 – TEST 0

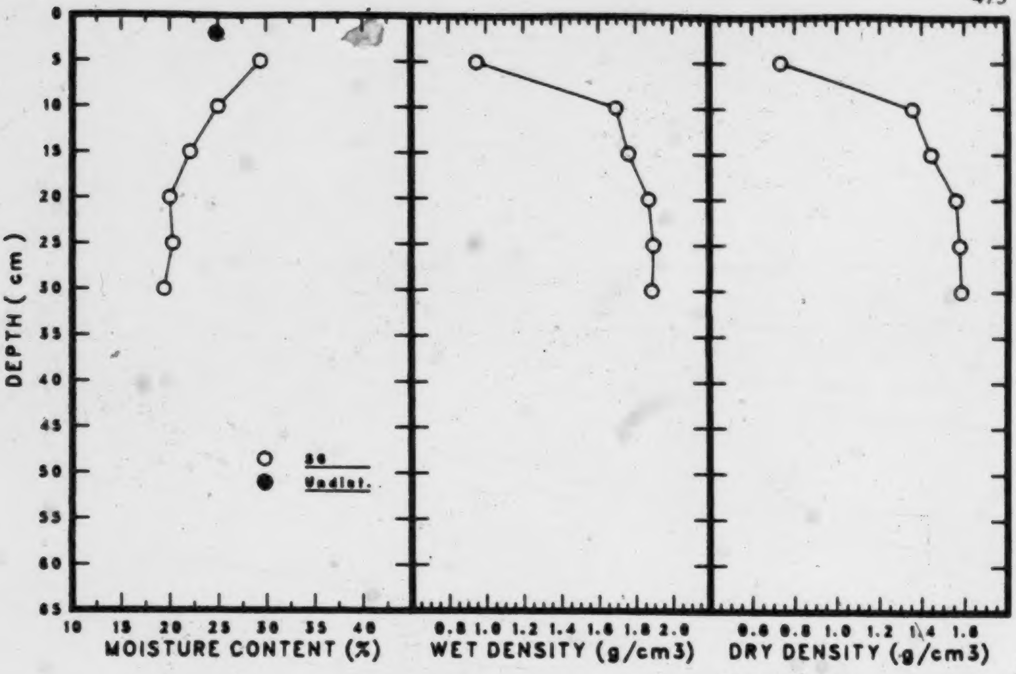


FIG. C.17 – Strata gauge measurements at SG 08 – TEST 1

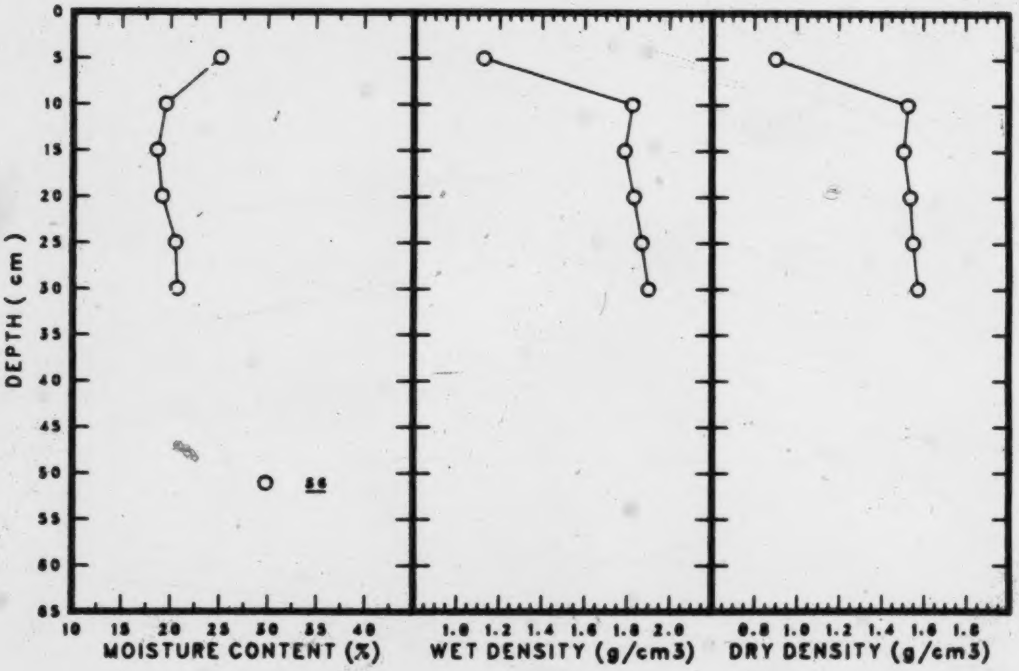


FIG. C.18 – Strata gauge measurements at SG 14 – TEST 1

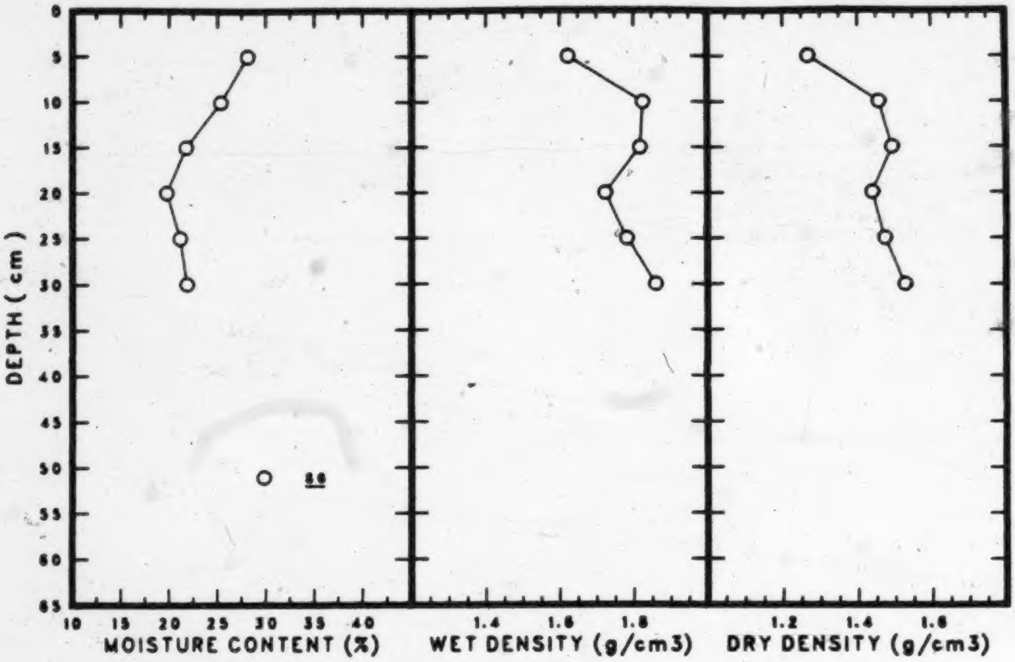


FIG. C.19 – Strata gauge measurements at SG 21 – TEST 1

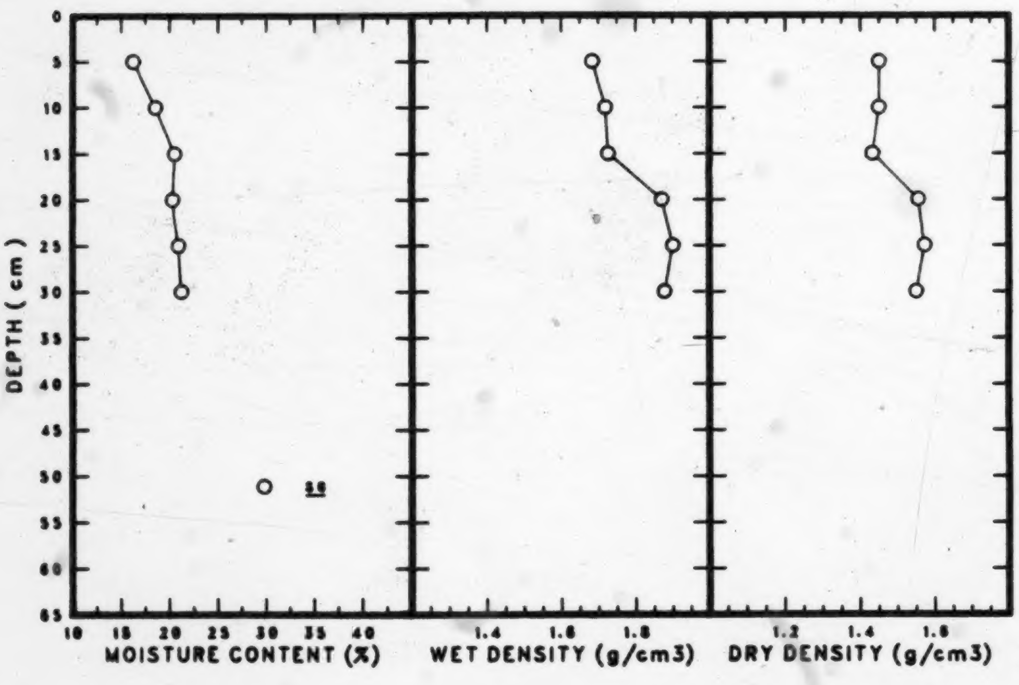


FIG. C.20 – Strata gauge measurements at SG 22 – TEST 1

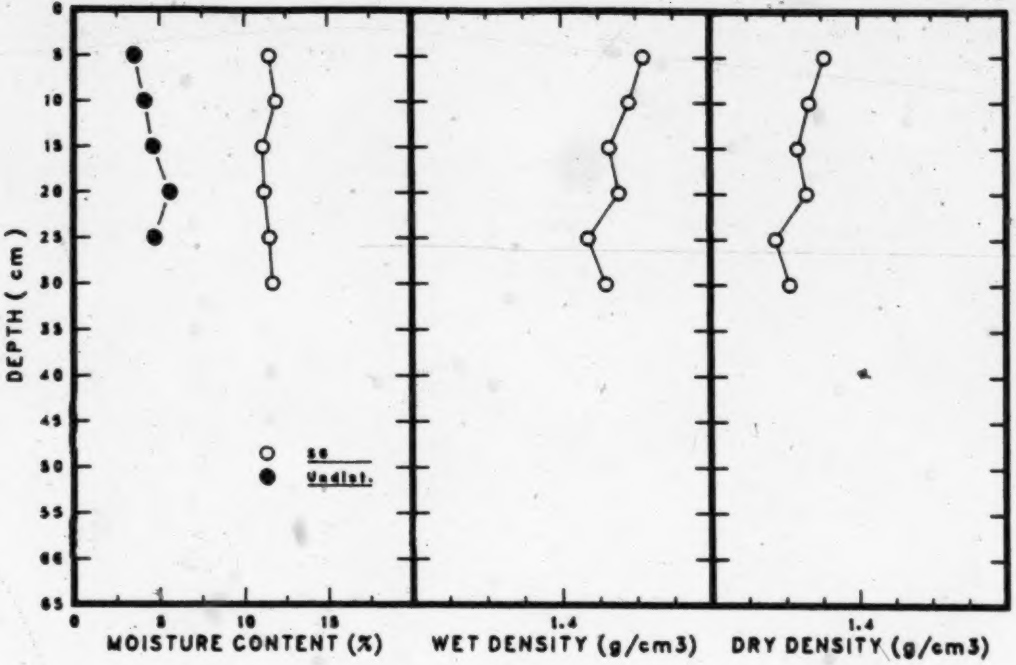


FIG. C.21 – Strata gauge measurements at SG 37 – TEST 1

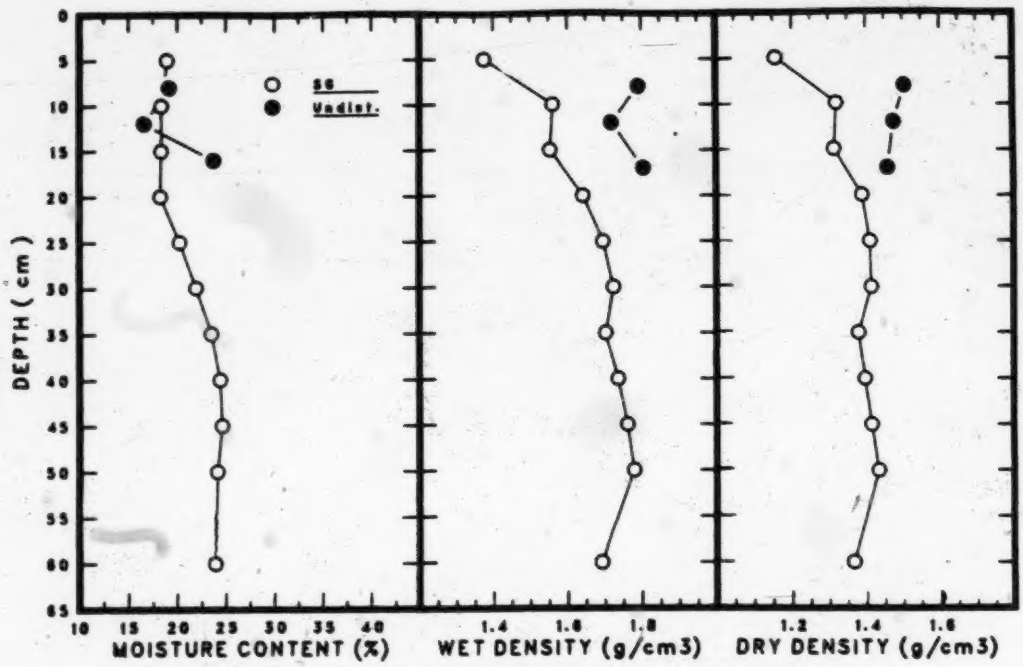


FIG. C.22 – Strata gauge measurements at SG 1 – TEST 2

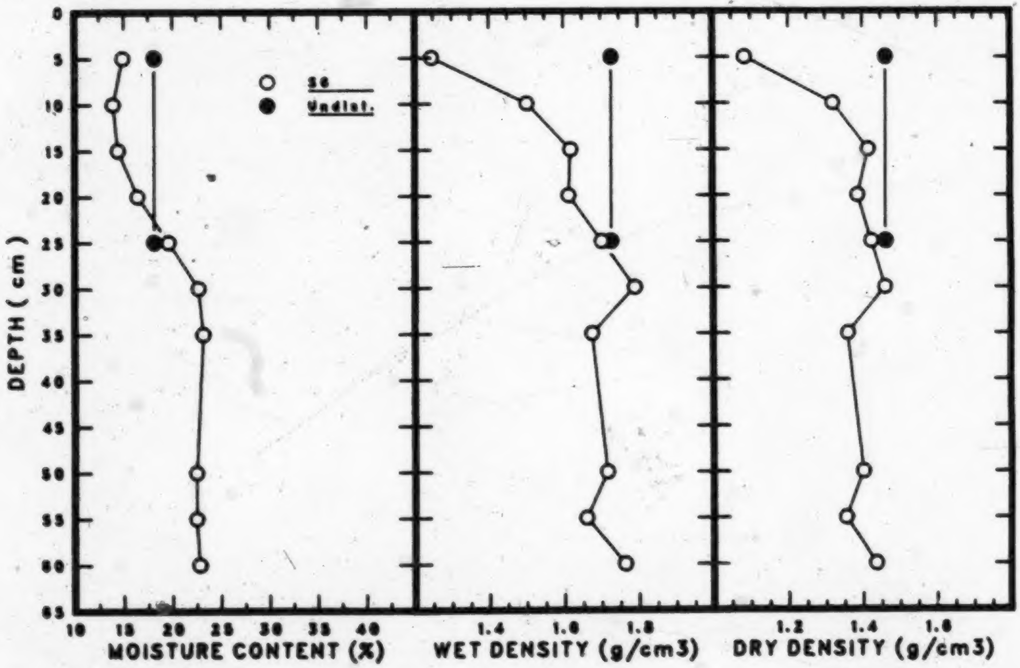


FIG. C.23 – Strata gauge measurements at SG 2 – TEST 2

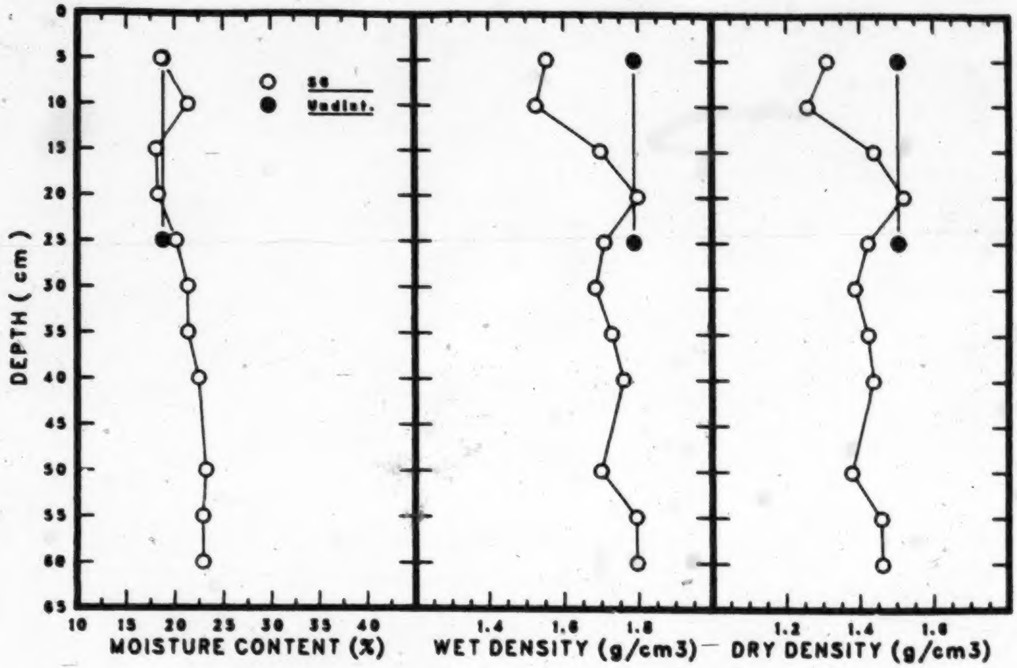


FIG. C.24 – Strata gauge measurements at SG 3 – TEST 2

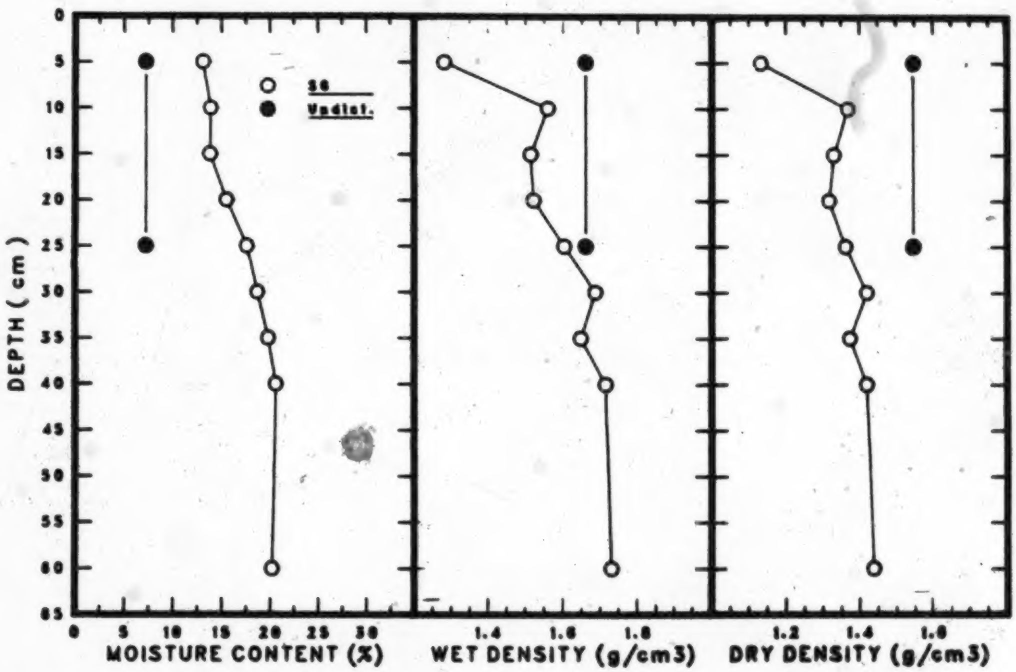


FIG. C.25 – Strata gauge measurements at SG 4 – TEST 2

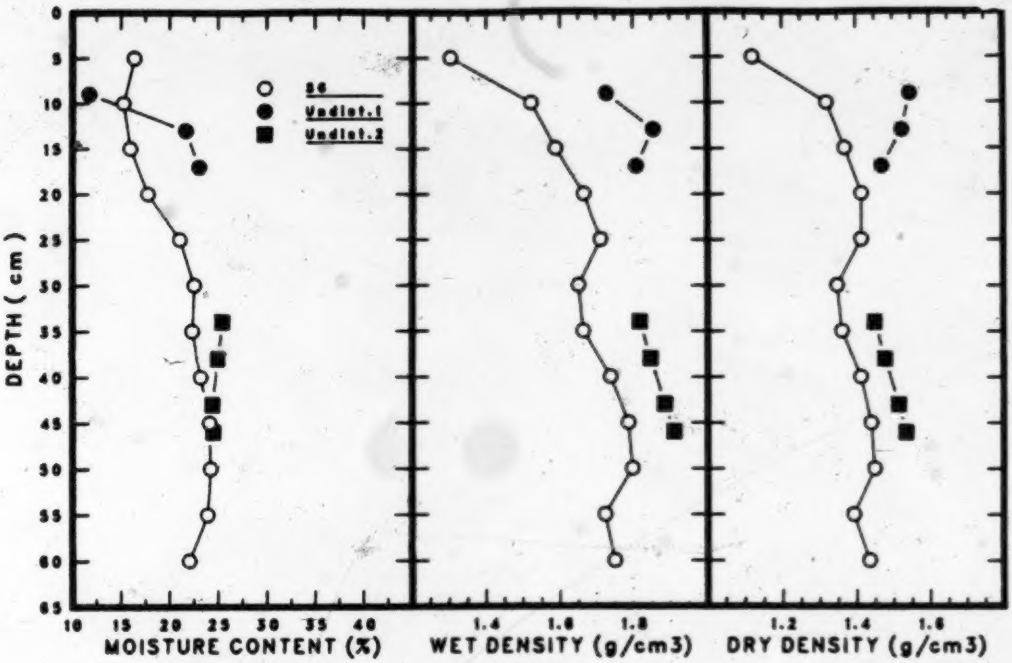


FIG. C.26 – Strata gauge measurements at SG 5 – TEST 2

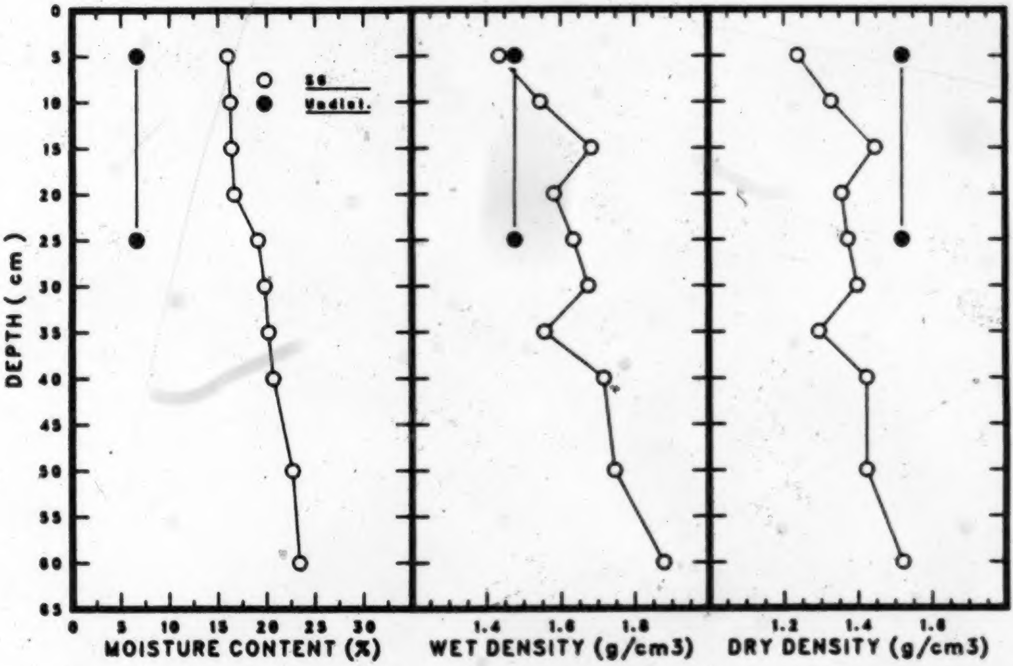


FIG. C.27 – Strata gauge measurements at SG 6 – TEST 2

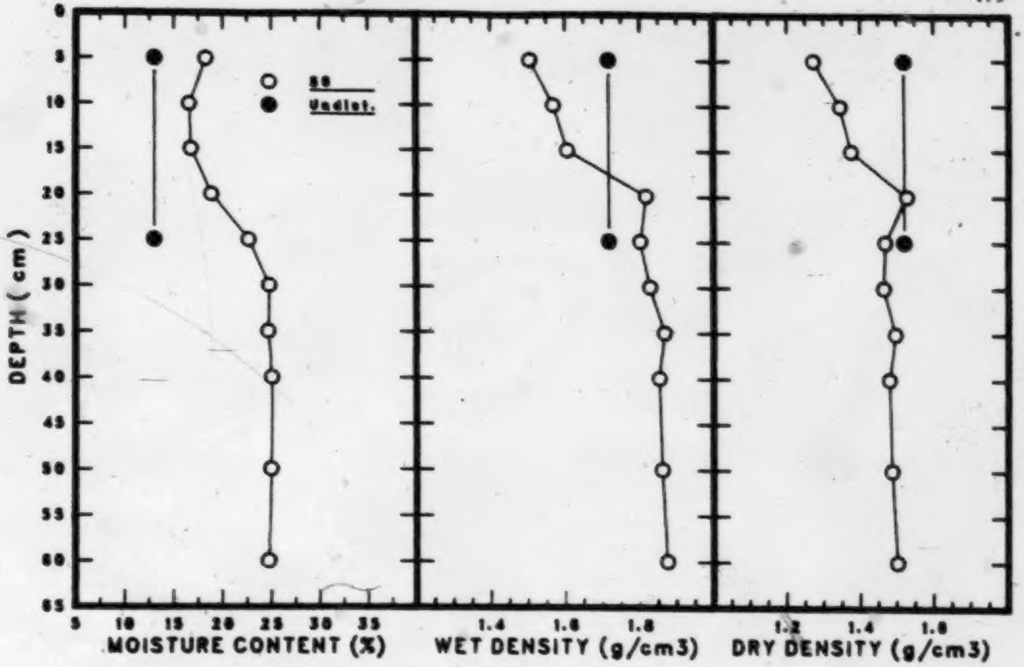


FIG. C.28 – Strata gauge measurements at SG 7 – TEST 2

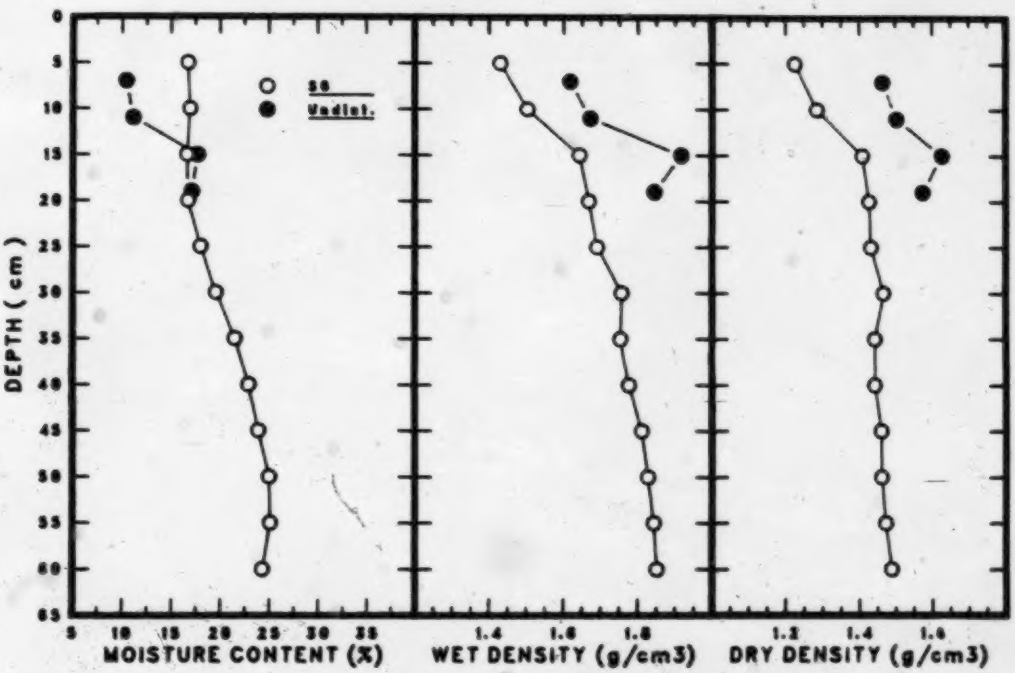


FIG. C.29 – Strata gauge measurements at SG 9 – TEST 2

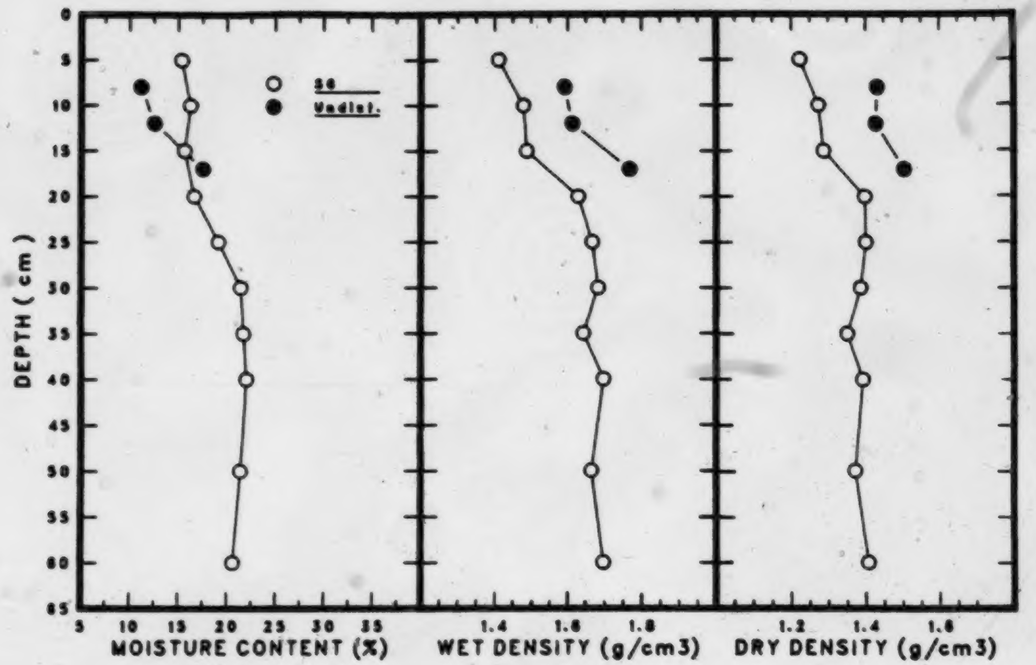


FIG. C.30 – Strata gauge measurements at SG 10 – TEST 2

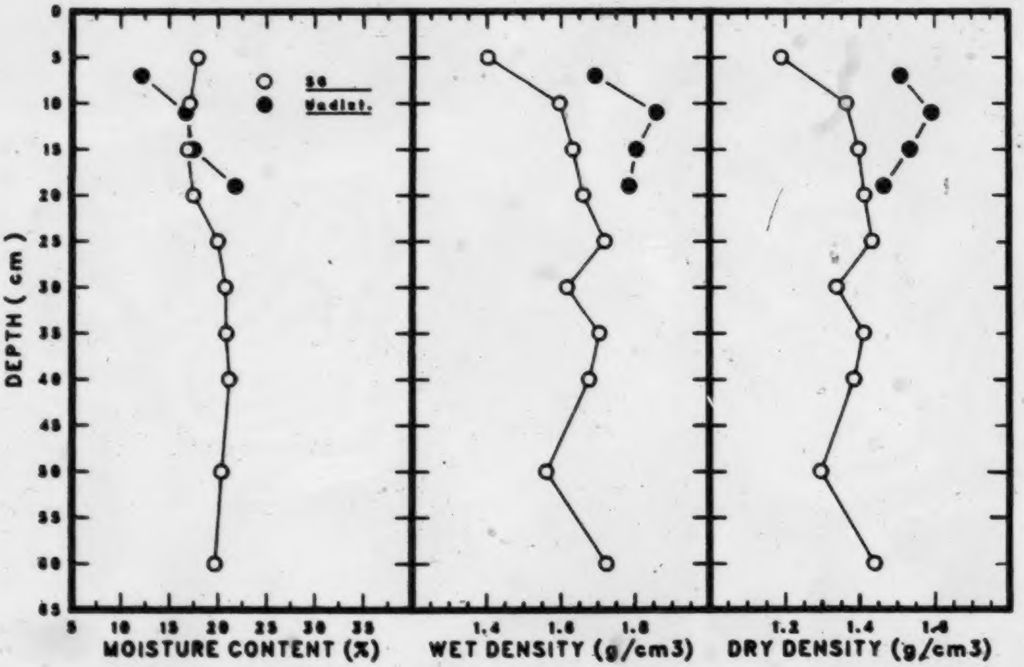


FIG. C.31 – Strata gauge measurements at SG 11 – TEST 2

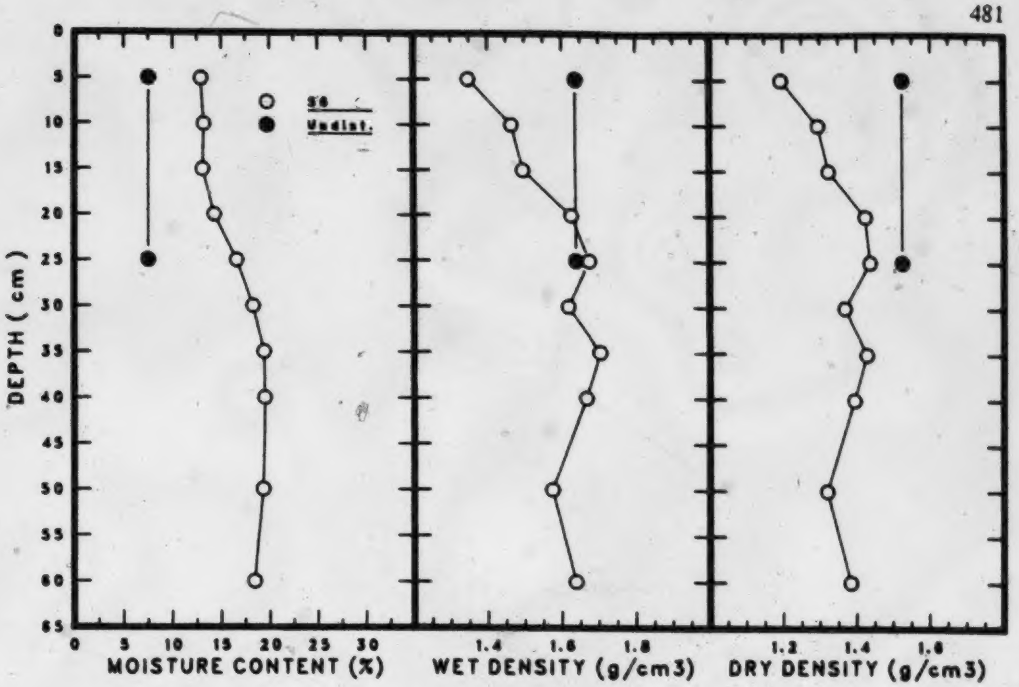


FIG. C.32 – Strata gauge measurements at SG 12 – TEST 2

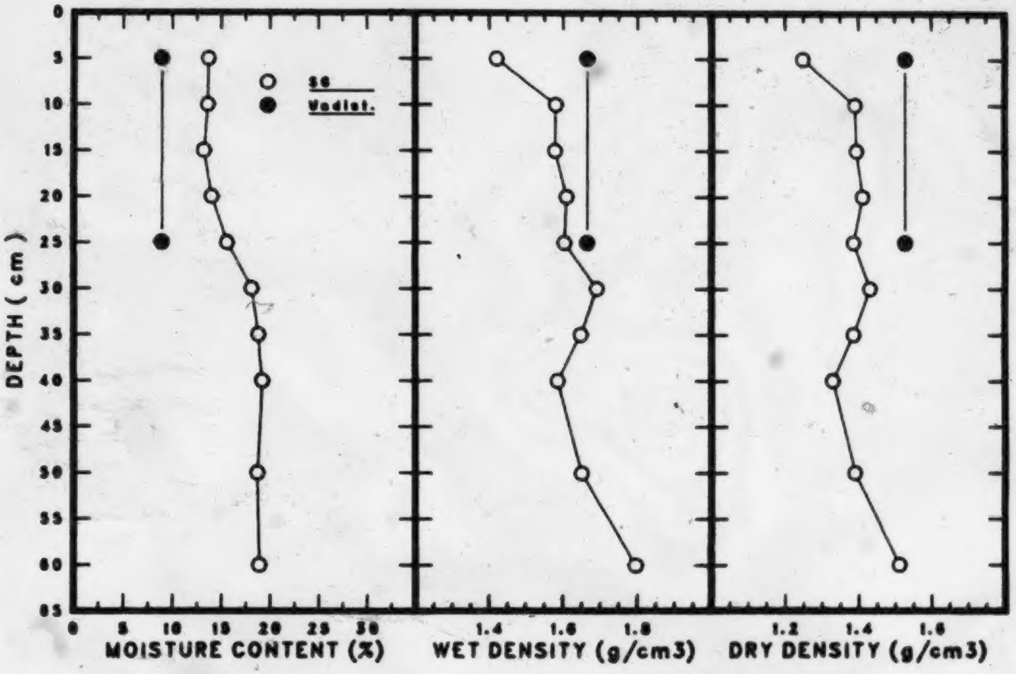


FIG. C.33 – Strata gauge measurements at SG 13 – TEST 2

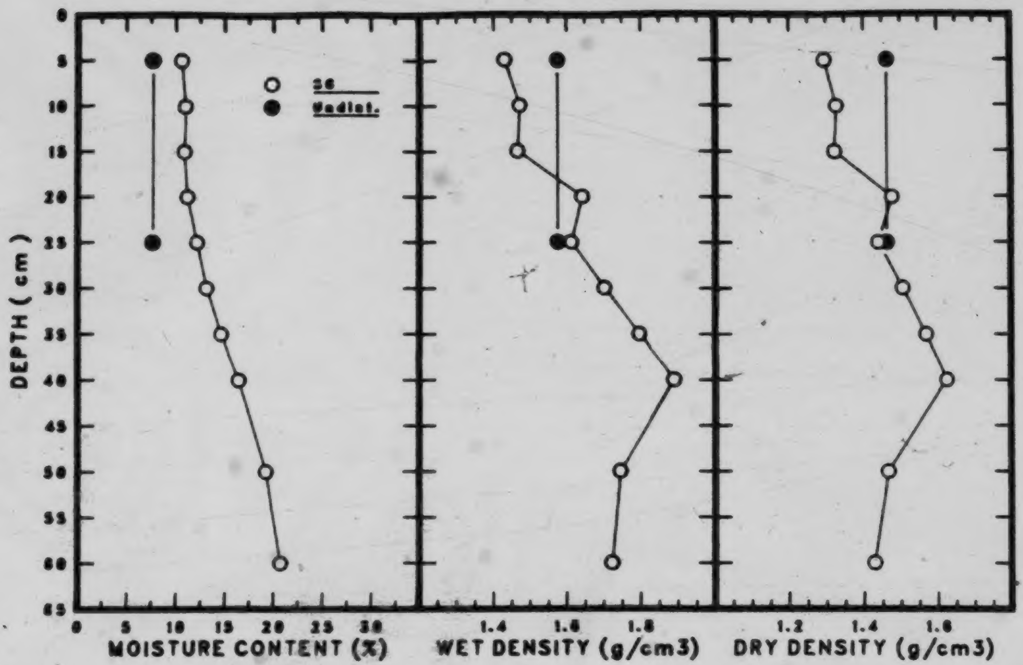


FIG. C.34 – Strata gauge measurements at SG 14 – TEST 2

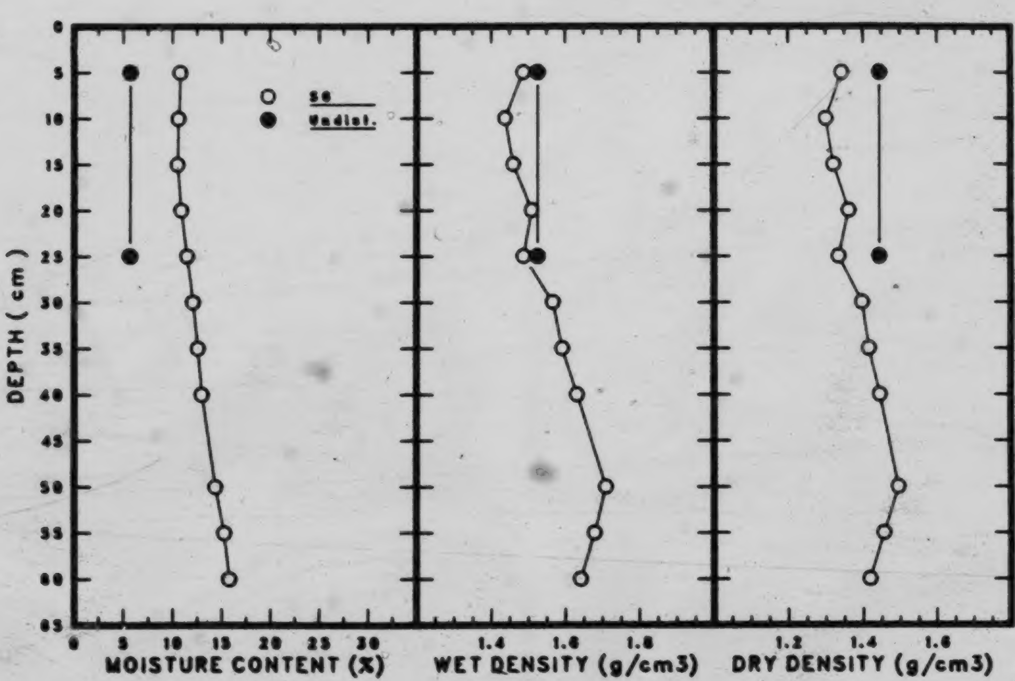


FIG. C.35 – Strata gauge measurements at SG 15 – TEST 2

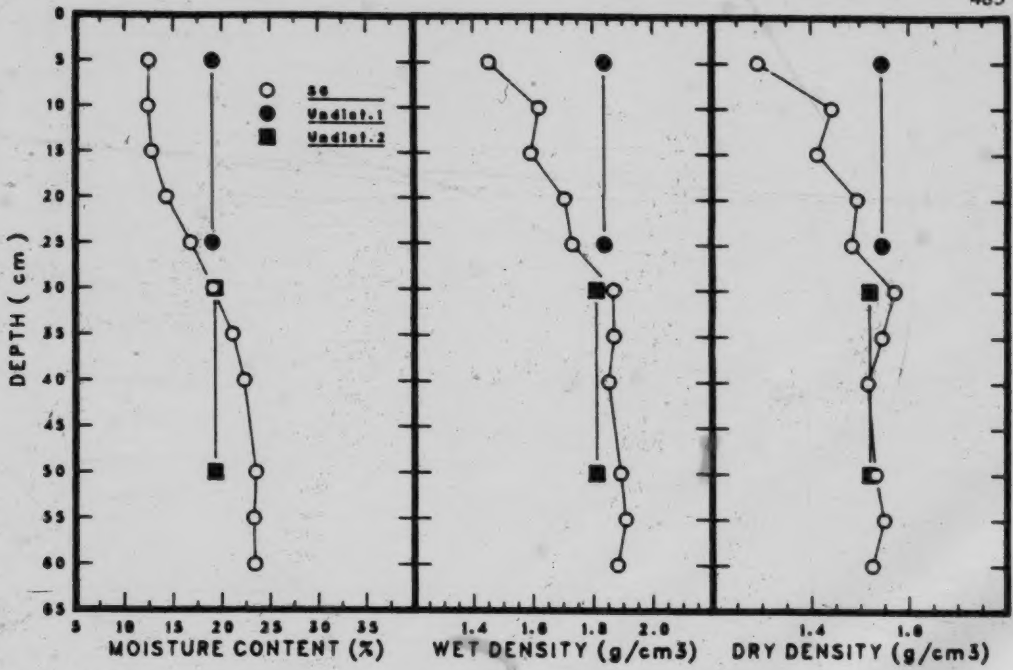


FIG. C.36 – Strata gauge measurements at SG 16 – TEST 2

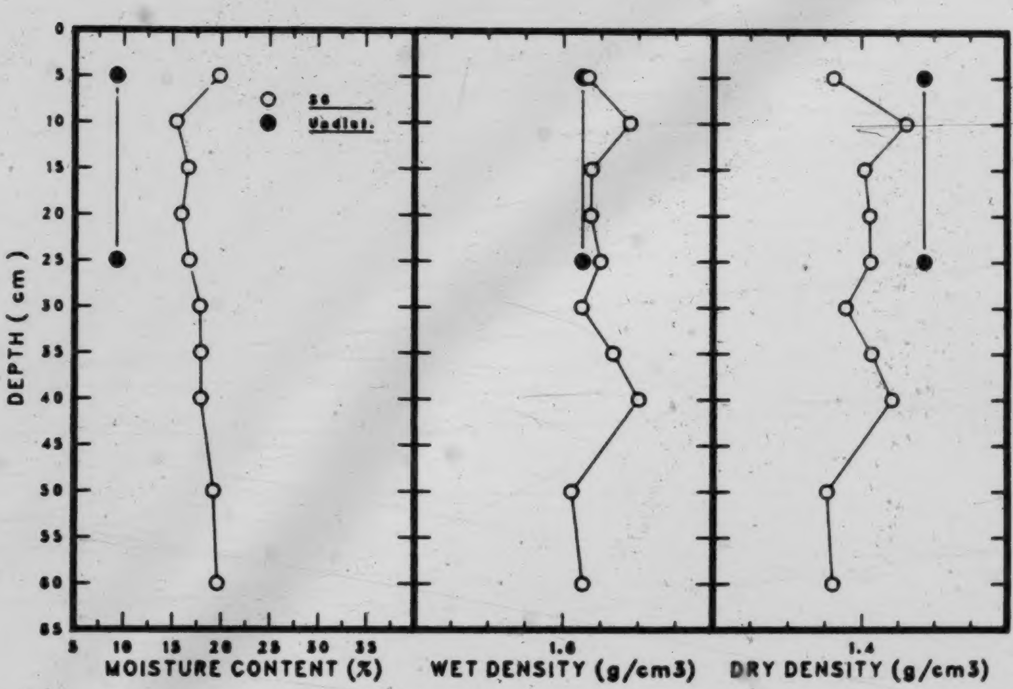


FIG. C.37 – Strata gauge measurements at SG 17 – TEST 2

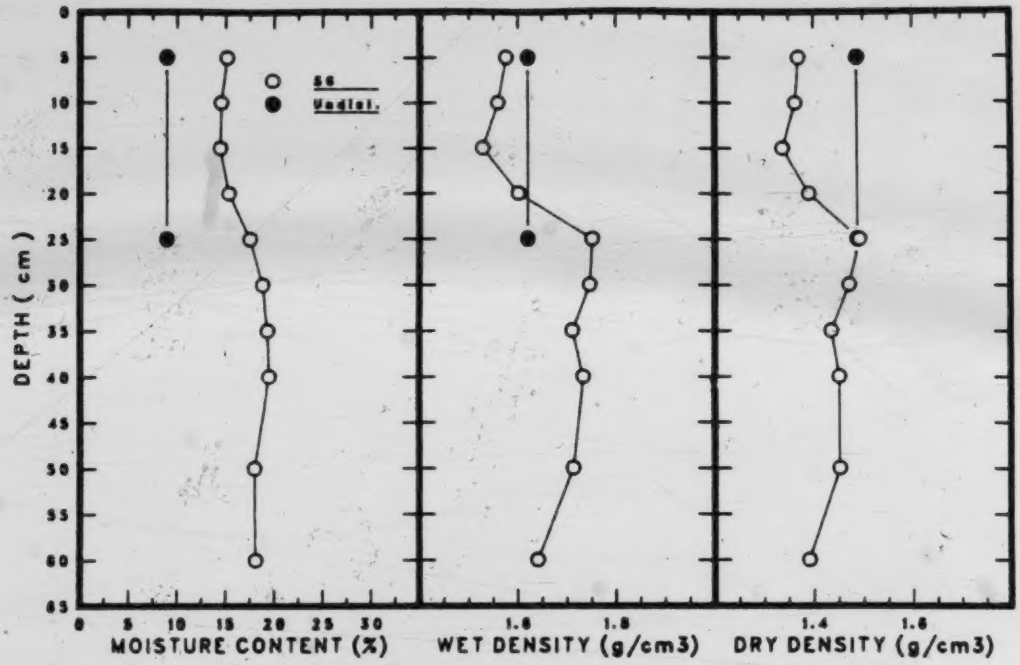


FIG. C.38 – Strata gauge measurements at SG 18 – TEST 2

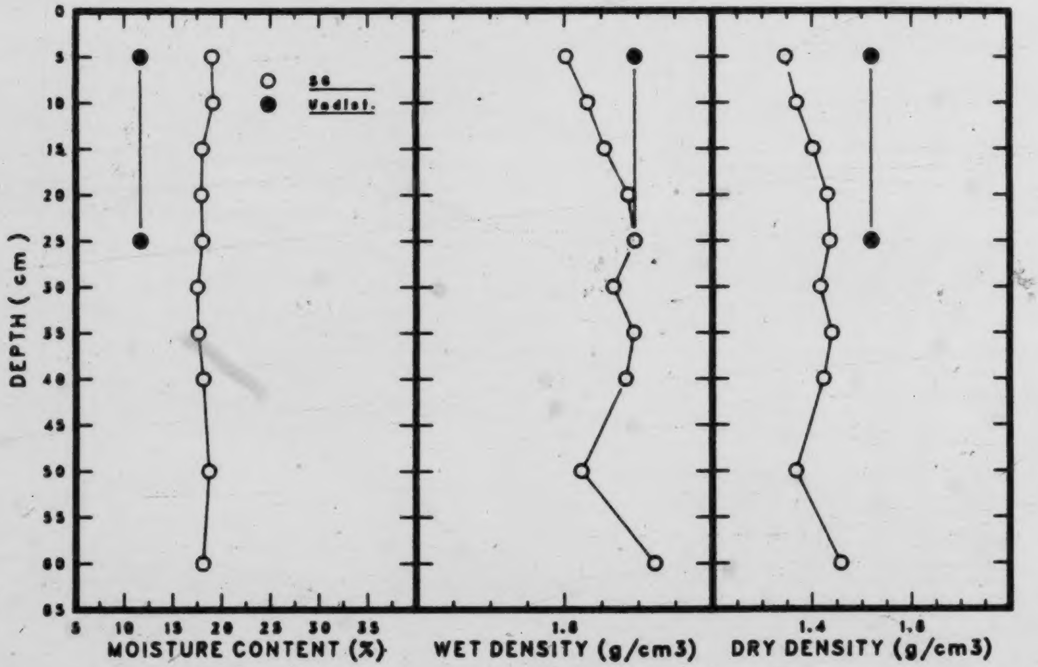


FIG. C.39 – Strata gauge measurements at SG 19 – TEST 2

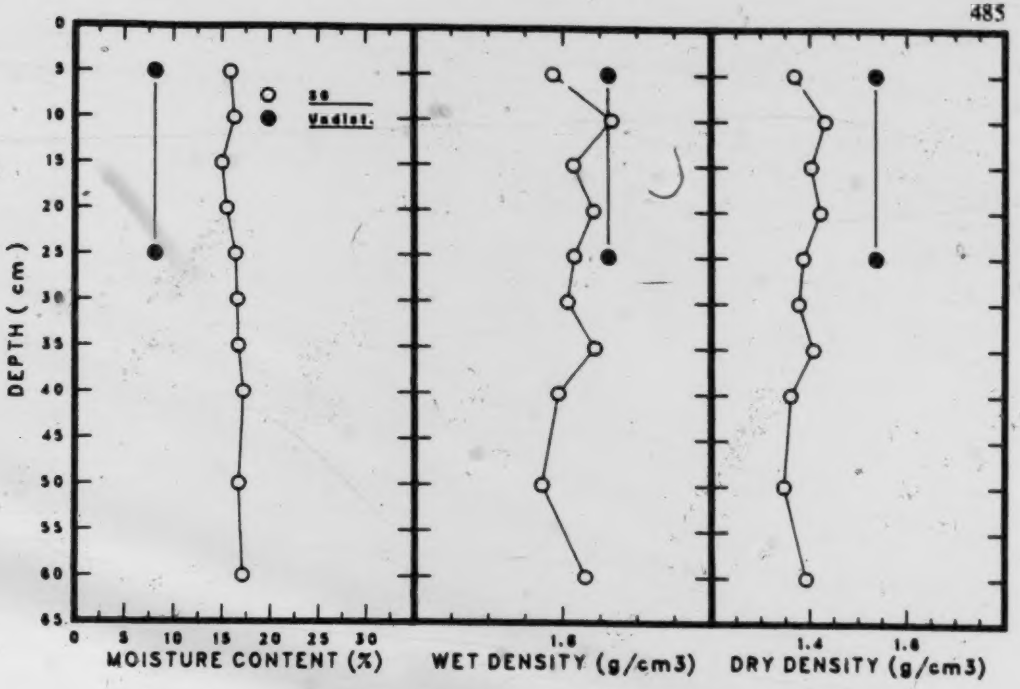


FIG. C.40 – Strata gauge measurements at SG 20 – TEST 2

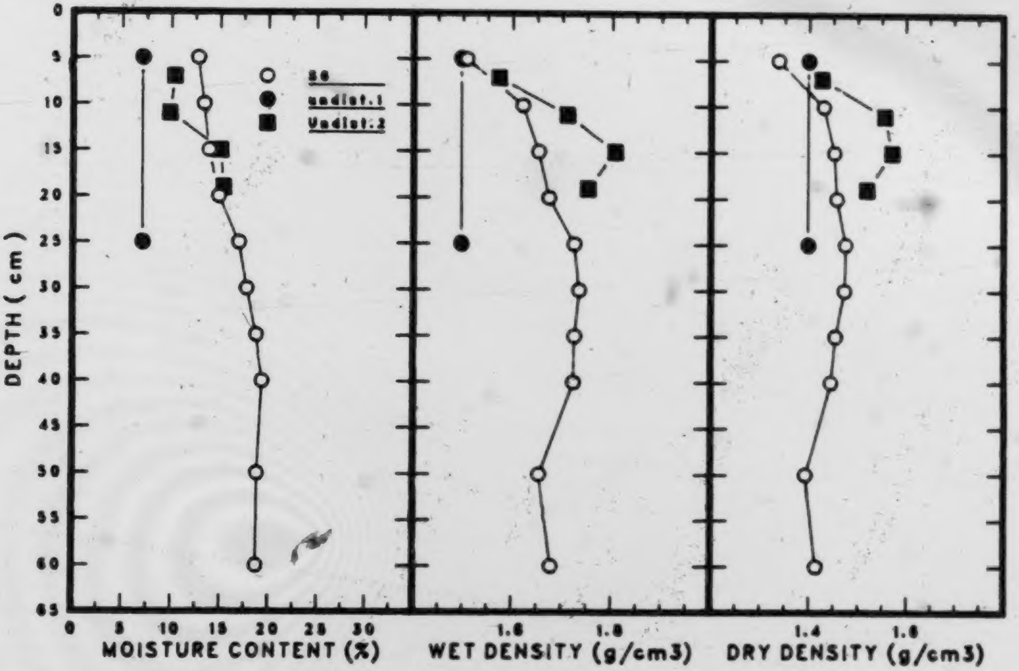


FIG. C.41 – Strata gauge measurements at SG 21 – TEST 2

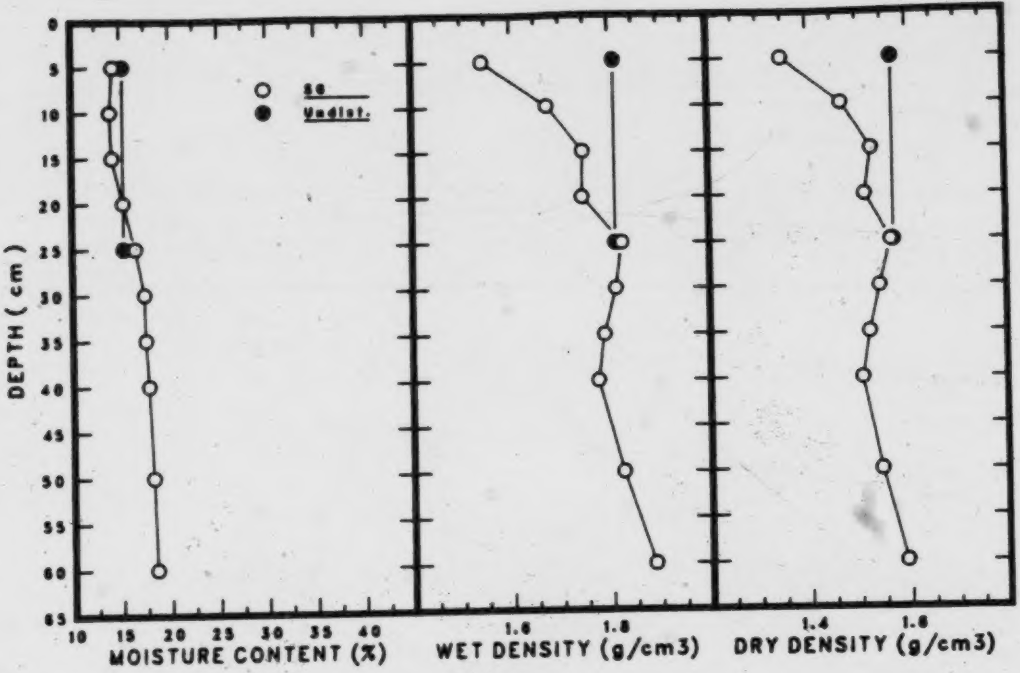


FIG. C.42 – Strata gauge measurements at SG 1 – TEST 4

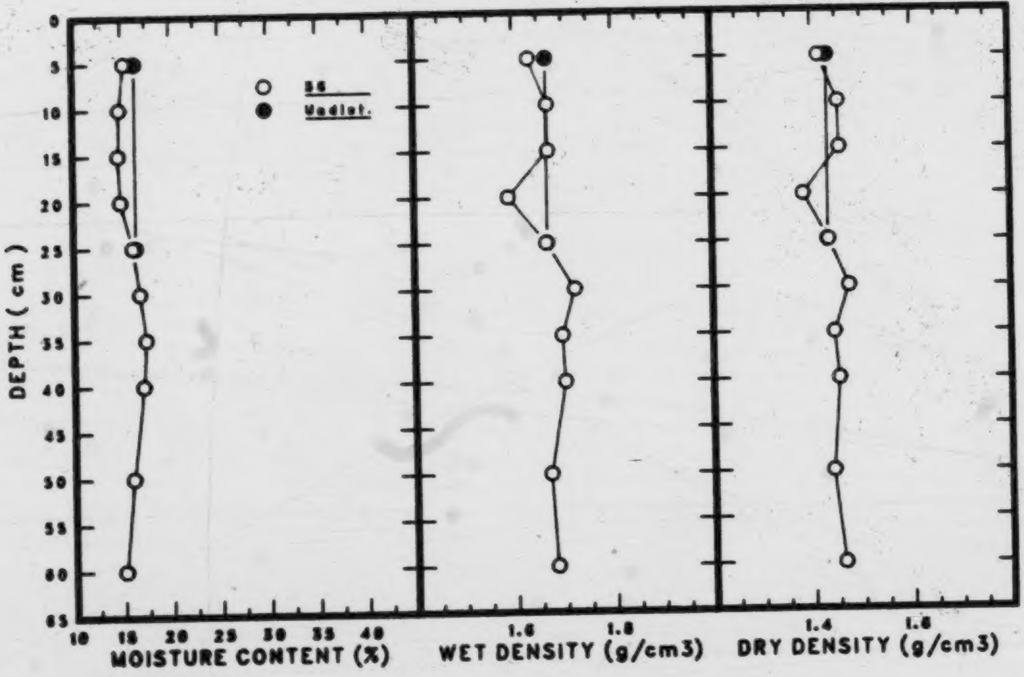


FIG. C.43 – Strata gauge measurements at SG 2 – TEST 4

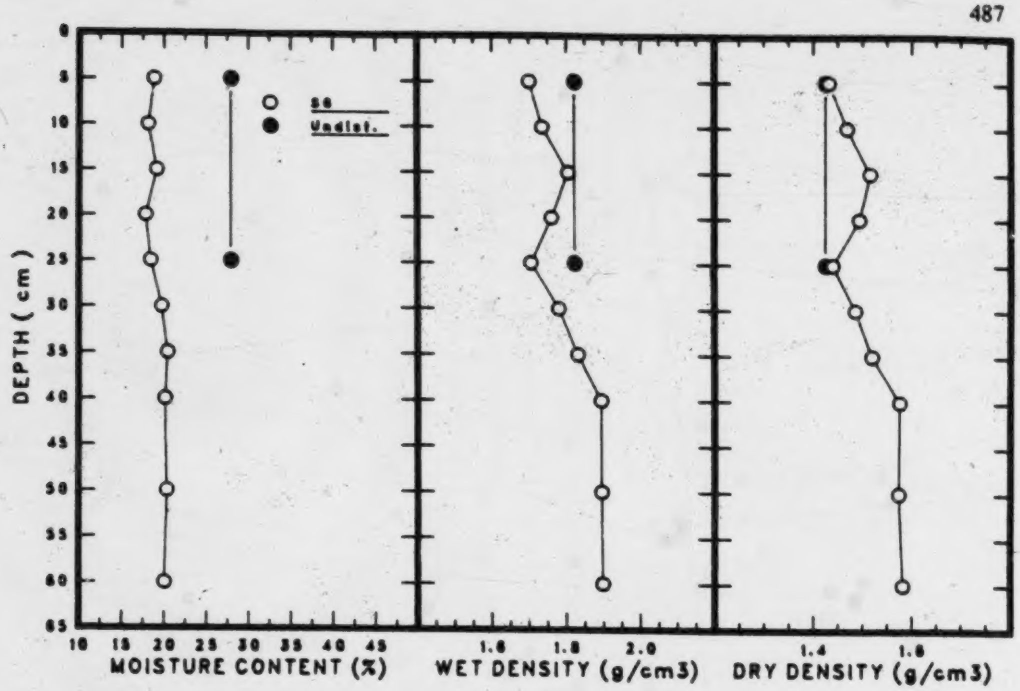


FIG. C.44 – Strata gauge measurements at SG 3 – TEST 4

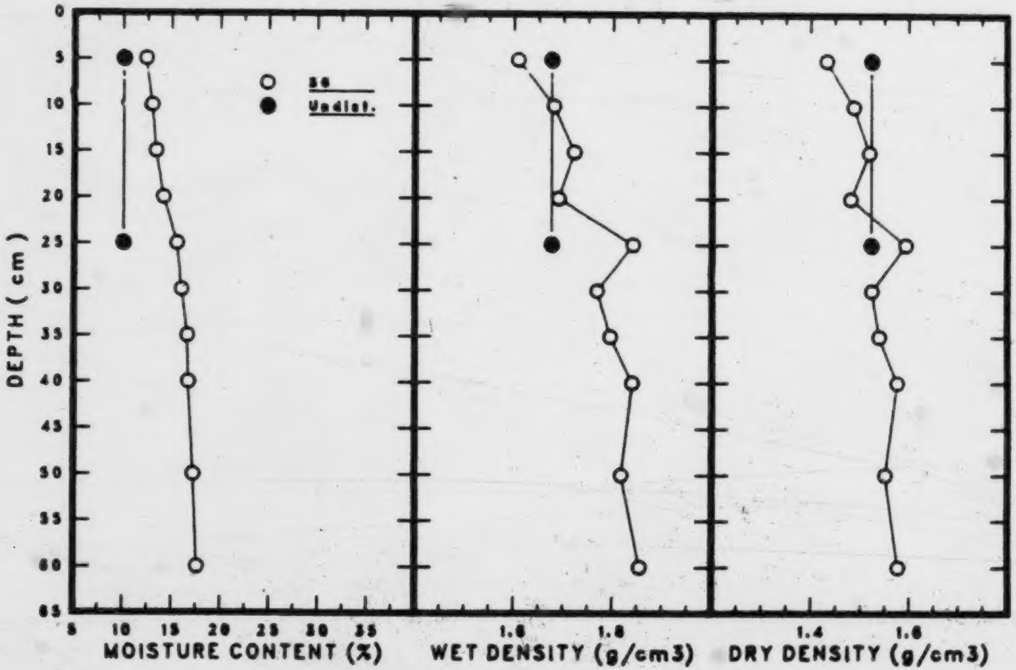


FIG. C.45 – Strata gauge measurements at SG 7 – TEST 4

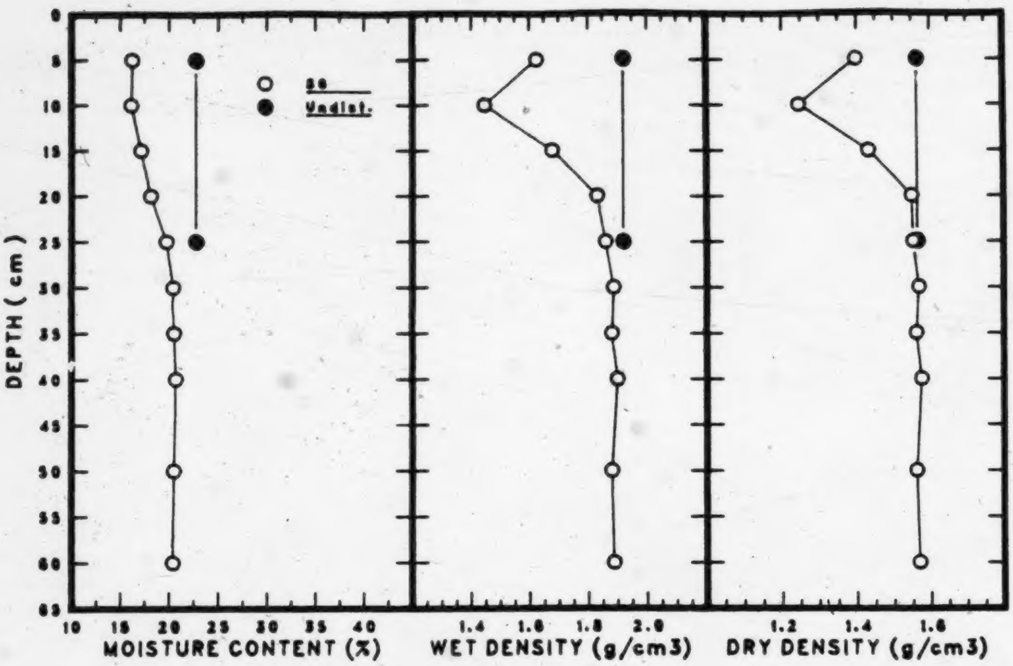


FIG. C.46 – Strata gauge measurements at SG 8 – TEST 4

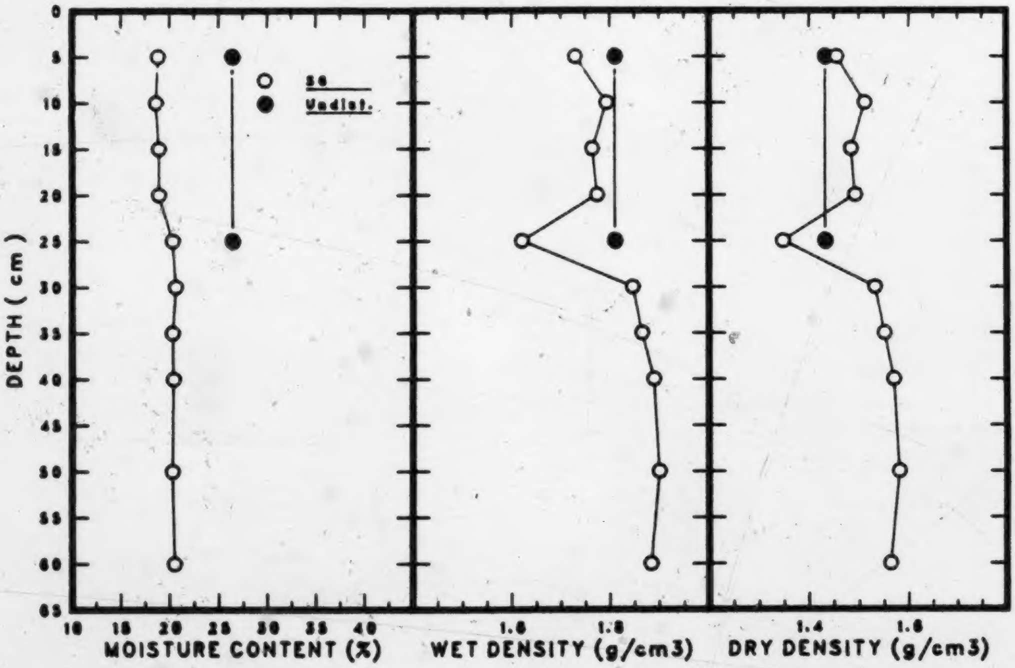


FIG. C.47 – Strata gauge measurements at SG 9 – TEST 4

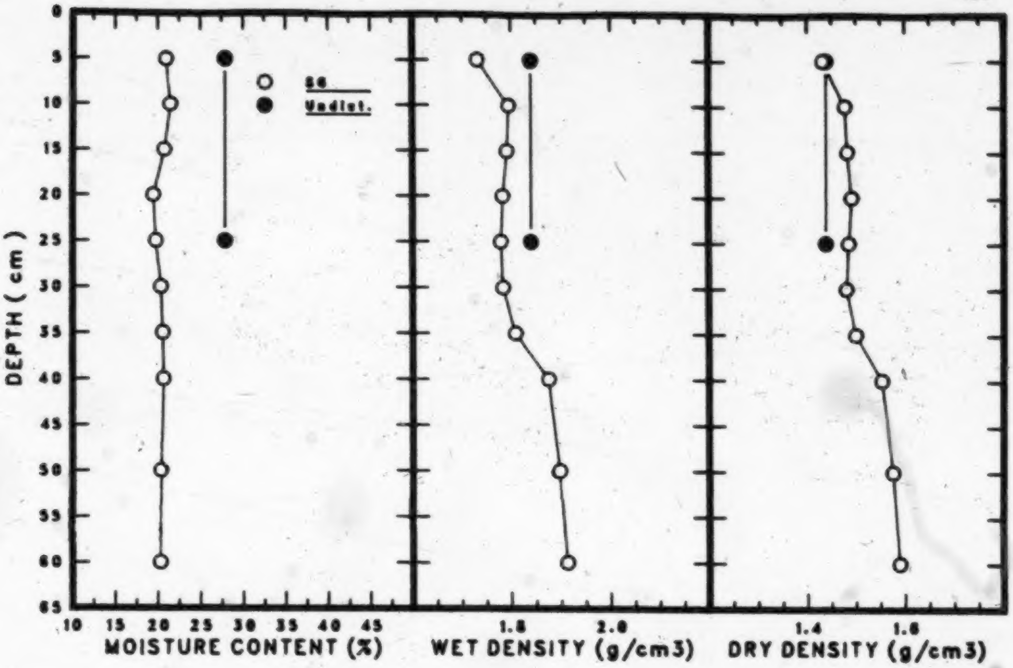


FIG. C.48 – Strata gauge measurements at SG 10 – TEST 4

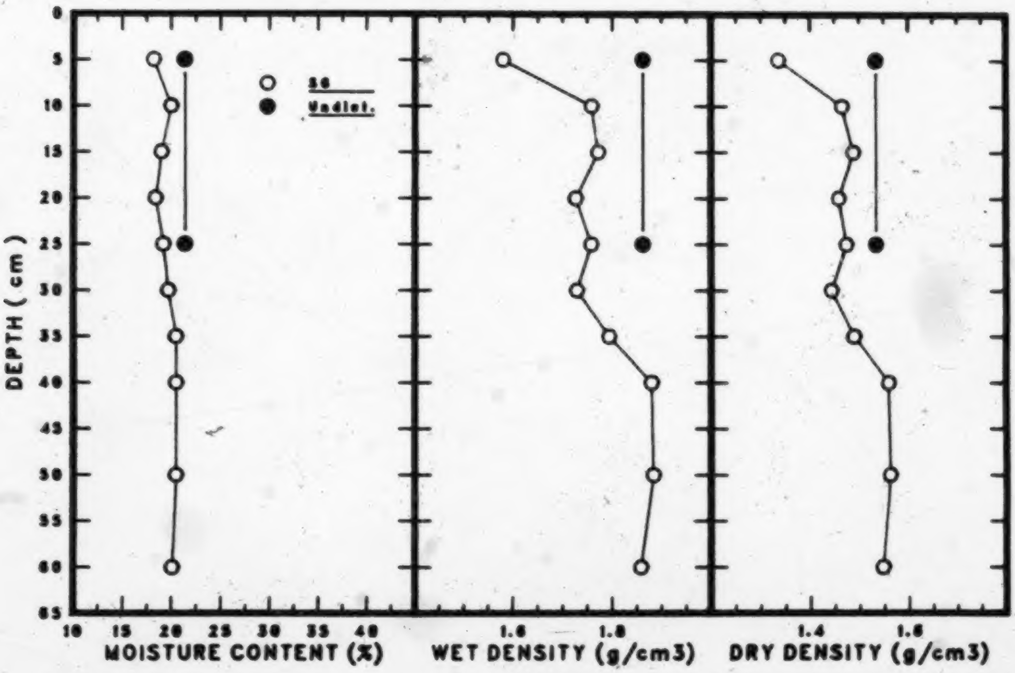


FIG. C.49 – Strata gauge measurements at SG 12 – TEST 4

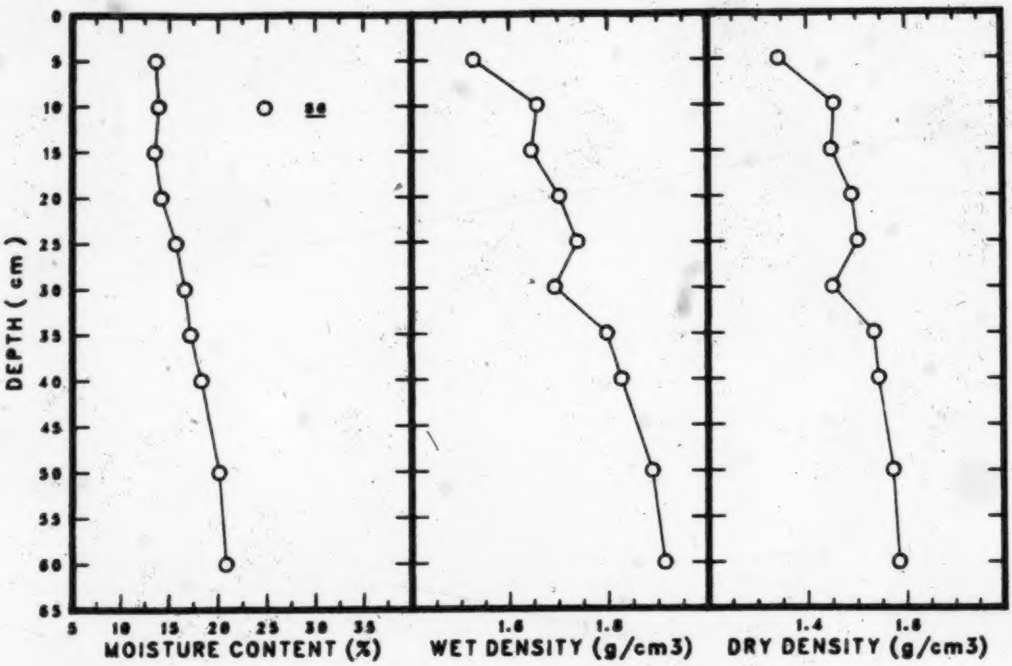


FIG. C.50 – Strata gauge measurements at SG 13 – TEST 4

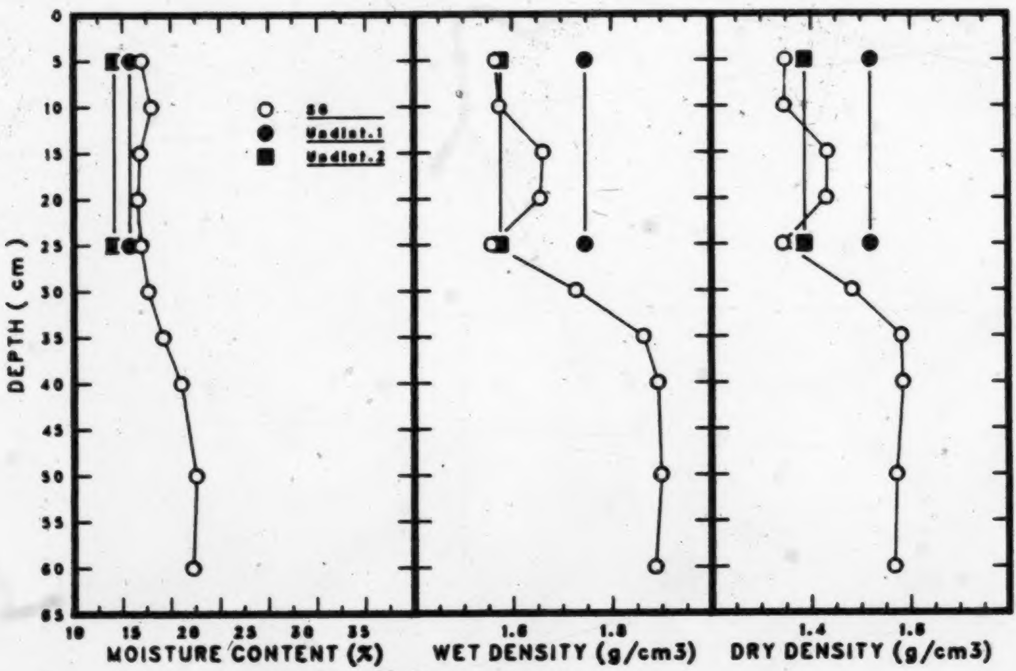


FIG. C.51 – Strata gauge measurements at SG 14 – TEST 4

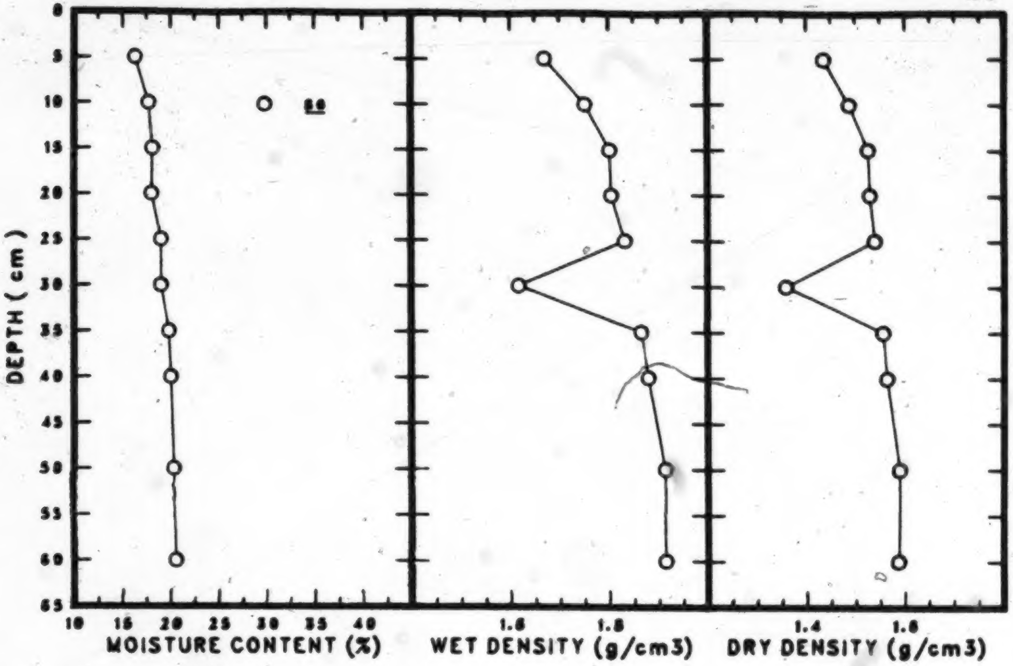


FIG. C.52 – Strata gauge measurements at SG 15 – TEST 4

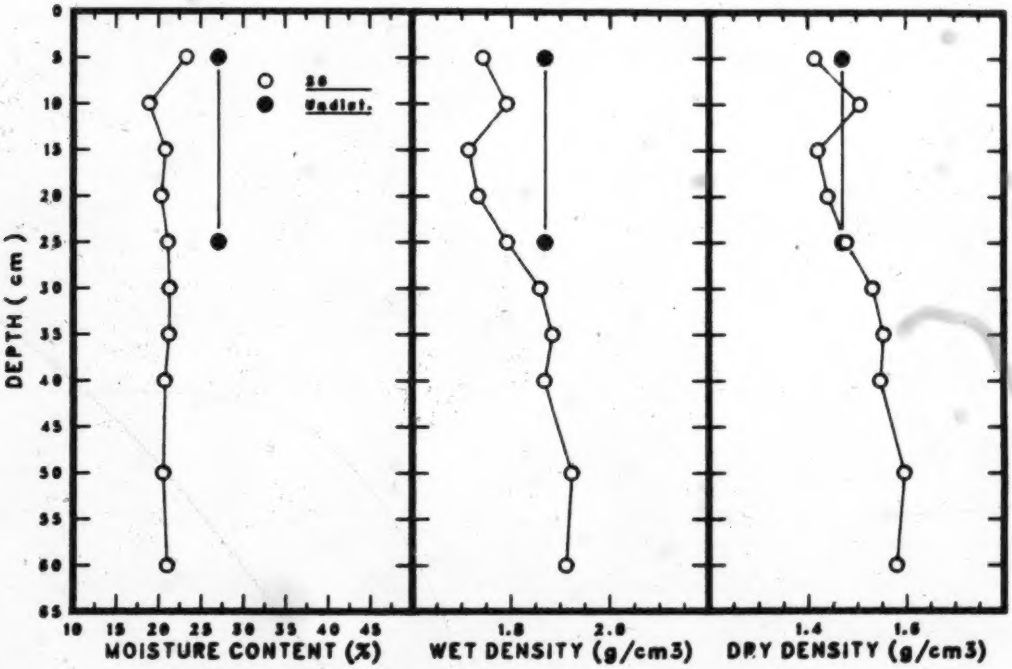


FIG. C.53 – Strata gauge measurements at SG 16 – TEST 4

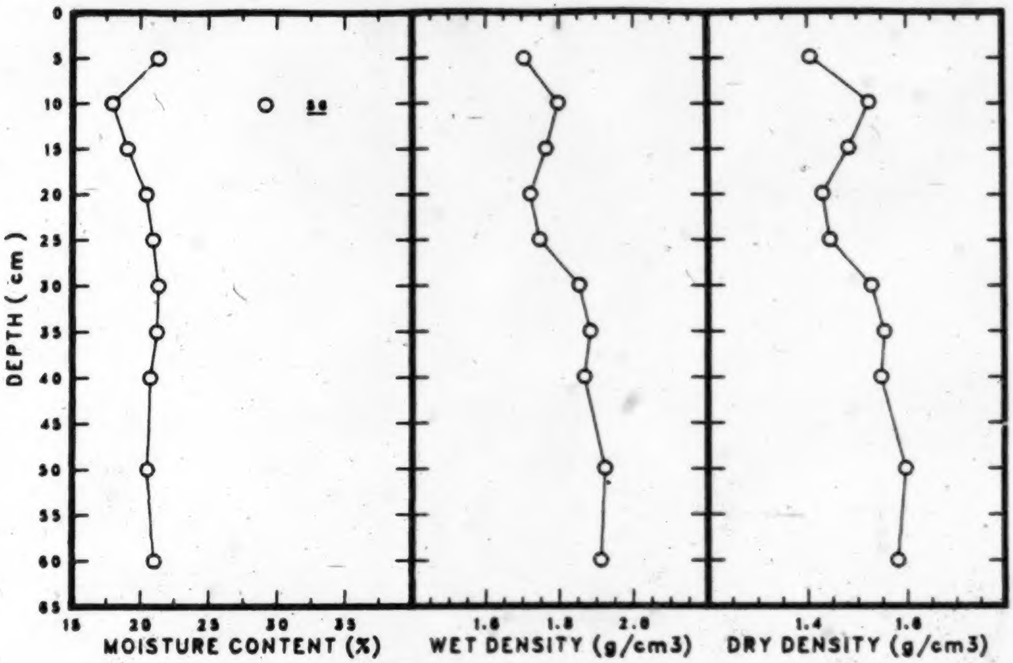


FIG. C.54 – Strata gauge measurements at SG 20 – TEST 4

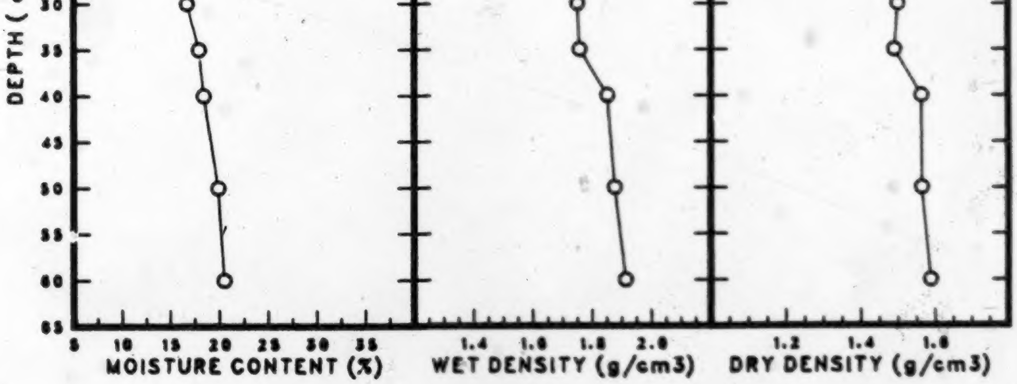


FIG. C.55 – Strata gauge measurements at SG 21 – TEST 4

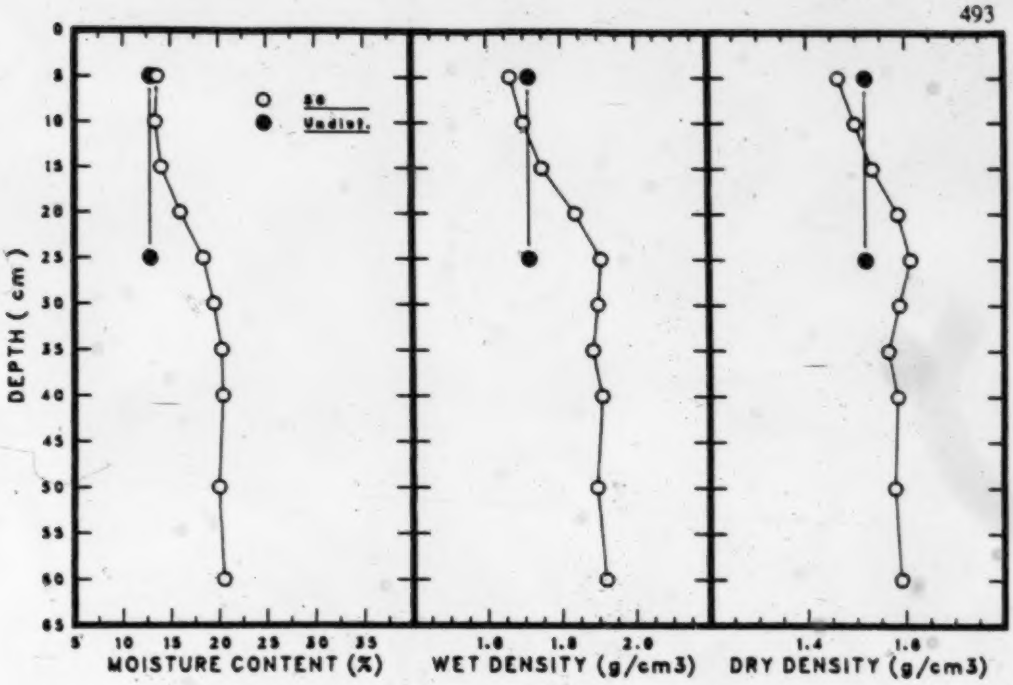


FIG. C.56 – Strata gauge measurements at SG 22 – TEST 4

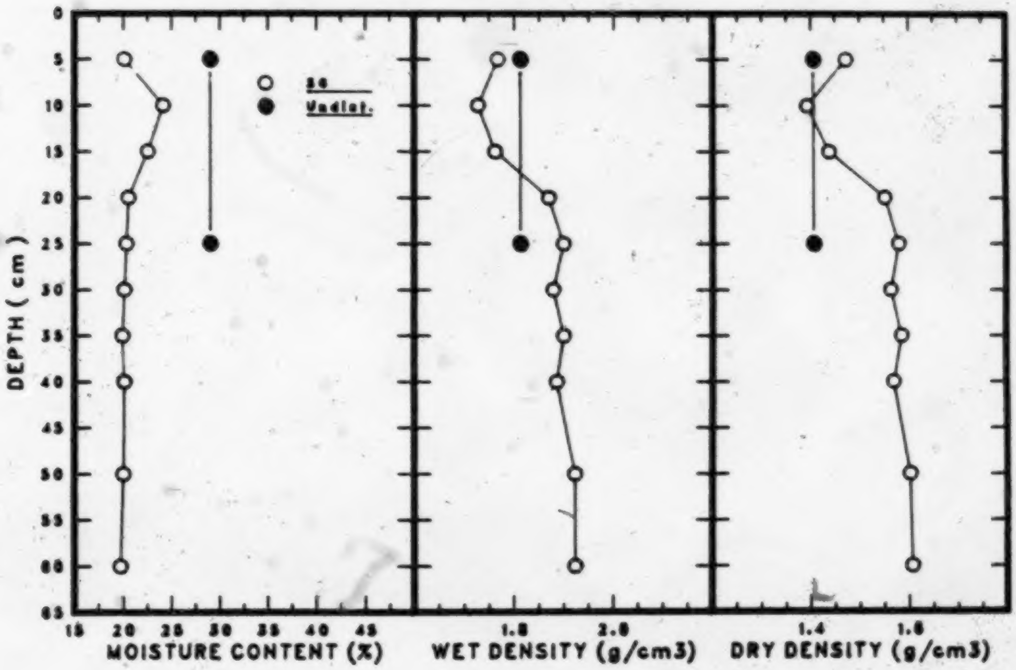


FIG. C.57 – Strata gauge measurements at SG 23 – TEST 4

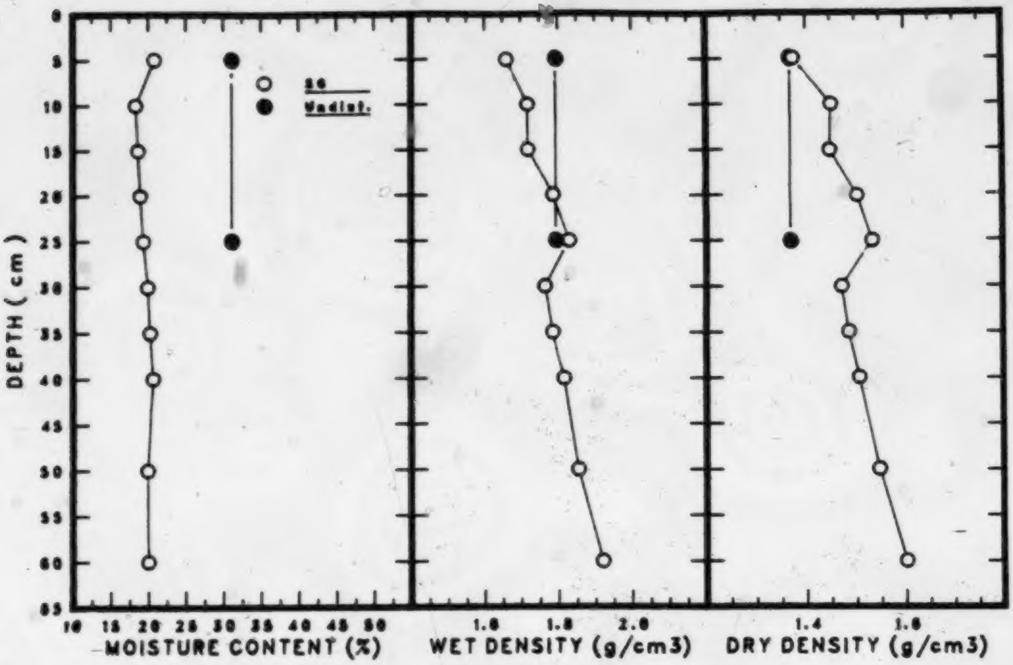


FIG. C.58 – Strata gauge measurements at SG 24 – TEST 4

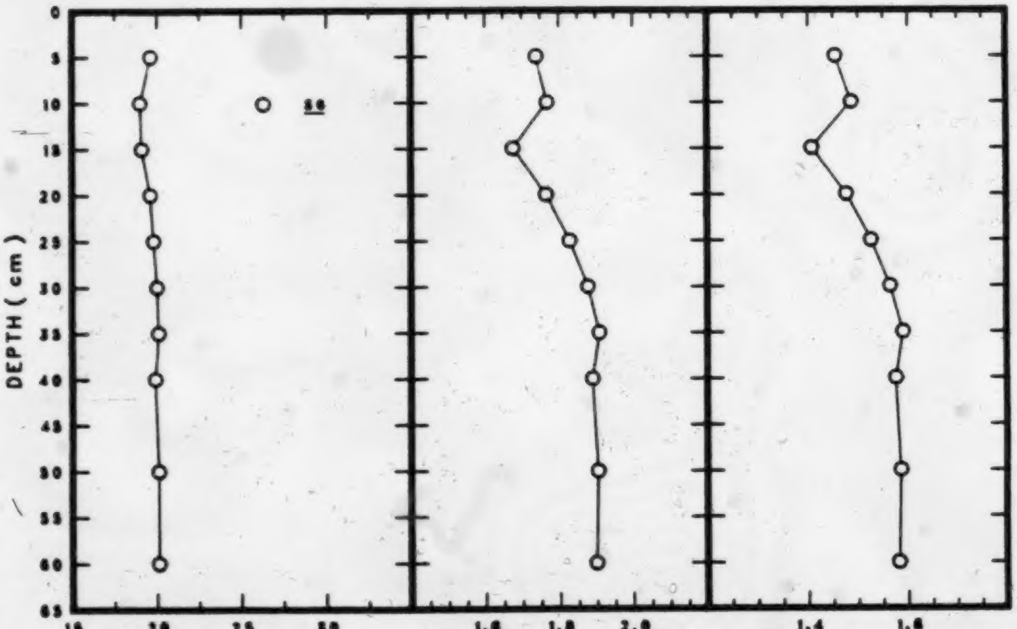


FIG. C.59 – Strata gauge measurements at SG 29 – TEST 4

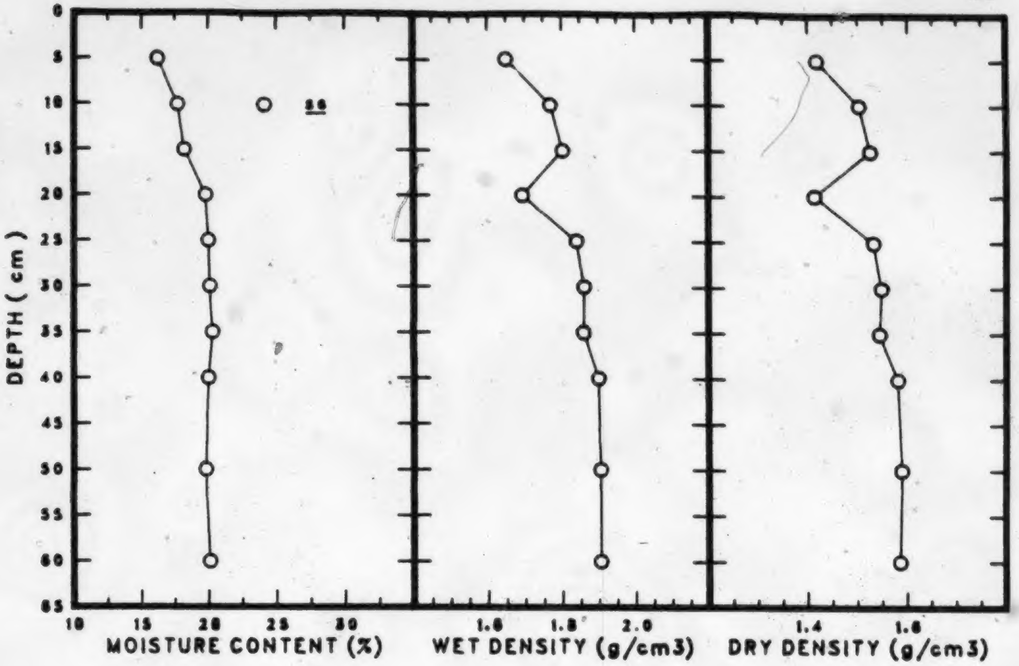


FIG. C.60 – Strata gauge measurements at SG 30 – TEST 4

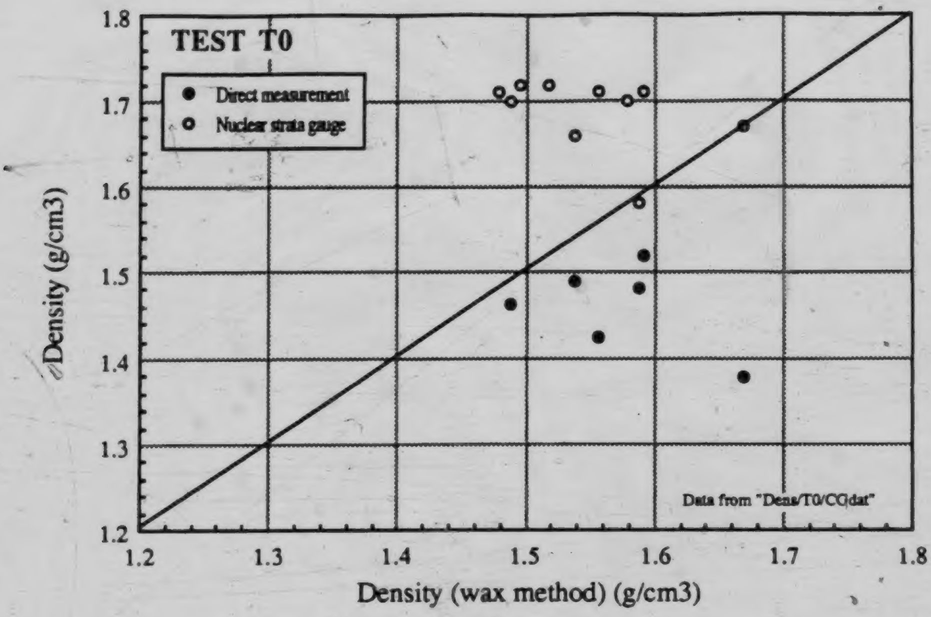


FIG. C.61 - Comparison between the three methods of measuring density (Test 0)

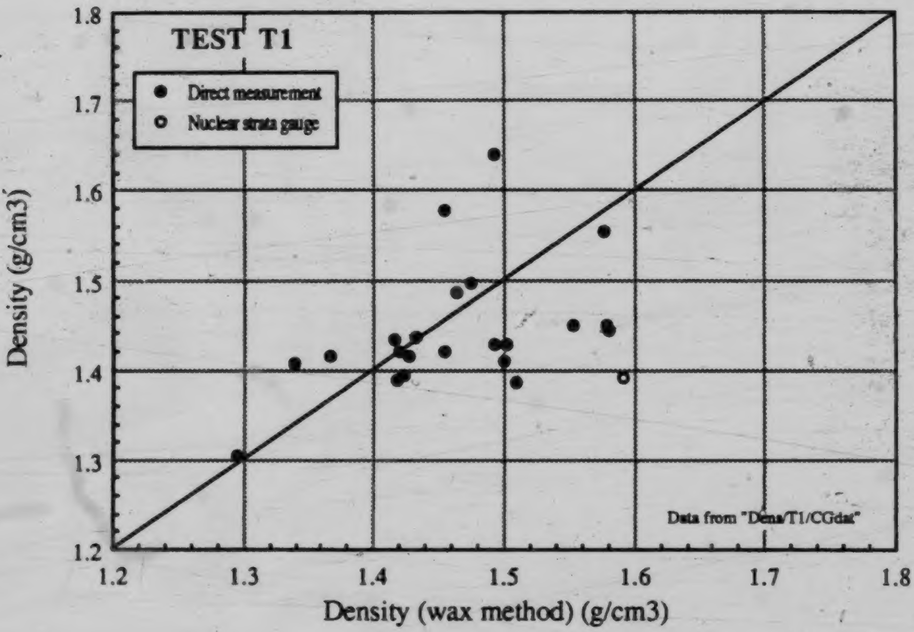


FIG. C.62 - Comparison between the three methods of measuring density (Test 1)

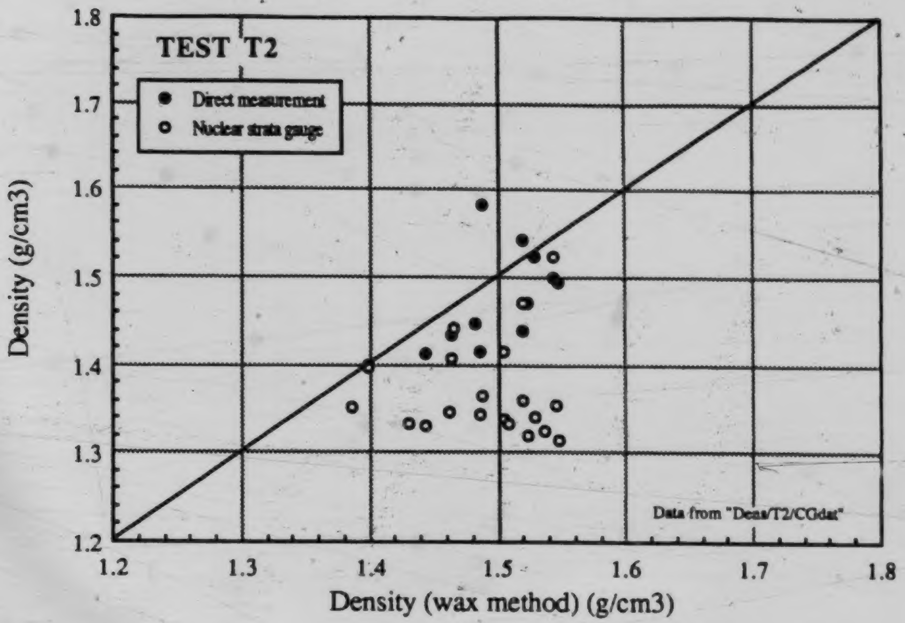


FIG. C.63 - Comparison between the three methods of measuring density (Test 2)

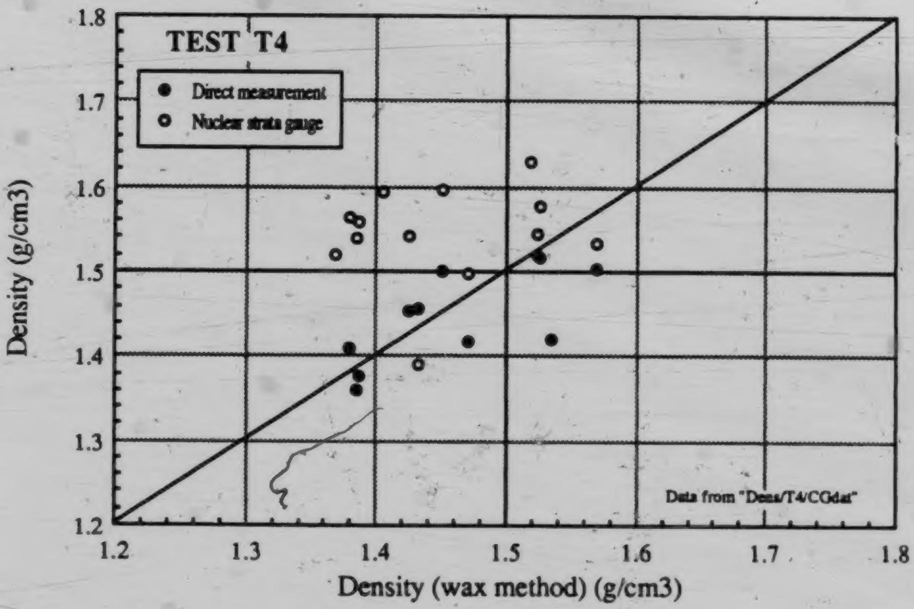


FIG. C.64 - Comparison between the three methods of measuring density (Test 4)

APPENDIX D - A STUDY ON RELATIVE DENSITY

D.1 - INTRODUCTION

Relative density is a standard way of defining the state of denseness of a granular material proposed by Terzaghi (1925). It is defined as :

$$RD = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{\rho_{max}}{\rho} \frac{(\rho - \rho_{min})}{(\rho_{max} - \rho_{min})}$$

where: ρ_{max} is the density obtained by a standard method of depositing sand in a dense state
ρ_{min} is the density obtained by a standard method of depositing sand in a loose state
ρ is the actual density of the sand sample

There are a few methods to determine maximum density (ρ_{max}) and minimum density (ρ_{min}) for granular materials. Different methods will provide different results because the efficiency of each method in obtaining the values of the reference densities varies from soil to soil. Factors such as gradation, grain size and shape, grain texture, amount and plasticity of fines affect the efficiency of each method in determining the absolute maximum and absolute minimum values of density. A perfect method of determining these absolute densities may not exist, and there will be always natural deposits with densities out of the limits determined in laboratory. For example: (a) eolian sands can have extremely low values of density which are not easily reproduced in the laboratory; (b) some interlocked sands can have extremely high in-situ densities due to diagenetic processes such as solution and crystal overgrowth, resulting in a grain imbrication that cannot be replicated in laboratory (Dusseault, 1977). The determination of the absolute minimum and maximum densities may not be even necessary. Relative density can be considered simply as a change in scale (from an absolute scale to a relative one) in order to make densities of different materials comparable supposedly in a more meaningful way. As such, "maximum" and "minimum" densities are simply reference values and whether they are the absolute maximum and minimum densities attainable becomes unimportant.

Besides the method of determination, there are several other factors that affect the values of the maximum and minimum densities. The most important factors are: particle shape, particle size, coefficient of uniformity, and amount and type of fines. There are several articles in the literature which discuss the effects of these parameters. However, how each parameter affects the reference densities varies from sand to sand. For this reason, a study was undertaken to determine the variations of the maximum and minimum densities of materials that were being used for hydraulic deposition research. The results of this study were compared to the published results and are presented below.

D.2 - METHODOLOGY ADOPTED

The standard specification ASTM 4254-83 was adopted for this study. A smaller mold than that specified in the ASTM standard (half of the volume) was used to determine the minimum density. The use of a smaller mold is considered to have a negligible influence on the results of minimum density because the particle size is very small compared to the mold size. Comparing results obtained using a smaller and the larger standard mold, Youd (1973) found consistent values for different sands. The dry method described in the ASTM standard was adopted for determination of the maximum density.

Two sands were used for this study. The first was tailings sand (TS sand) from the tailings dyke at Syncrude Canada Ltd., in Fort McMurray, Alberta, Canada. This is a uniform fine sand composed mainly of subangular to subrounded quartz grains. The other sand (KS sand) was a uniform medium quartz sand with subrounded grains used for hydraulic fill research by Küpper (1991). The range of values of the coefficient of uniformity C_U, mean grain size D_{50} and percentage of fines (< 74 μm) for these materials in-situ and in a laboratory flume (see Küpper, 1991) is presented on Table D.1.

A preliminary test series to study some of the important factors affecting ρ_{max} and ρ_{min} was carried out on artificially proportioned gradations of the TS sand. A large bulk sample of the tailings sand was sieved and the fractions were combined to form nine different samples with gradations as presented in Table D.2 and Figure D.1. Examples of actual grain size distribution curves are presented in Fig.D.2a. The values of C_U, D_{50} and percentage of fines for these nine artificial samples are presented in Table D.3. Soils 1 to 3 have the same C_U and percentage of fines but varying D_{50} in order to study the effect of particle size on ρ_{max} and ρ_{min}. Comparisons between soils 2 and 4 and among soils 3, 5 and 6 are expected to give information on the effect of C_U. Soils 7, 8 and 9 were selected with the objective of analyzing the effect of the amount of fines. It was not possible to keep D_{50} and C_U constant while varying the amount of fines, so it was decided to keep C_U constant and vary D_{50} as little as possible. The maximum amount of fines used was 15% in order to stay within the limits established by ASTM 4254-83 for applicability of the method.

A second series of tests was carried out using KS sand samples (Table D.4 and Figure D.2b). Since this is a commercial blasting sand, it is washed, sieved and cleaned of fines at the plant, using a process not very different from the one used to prepare the first nine TS samples.

Finally, seven TS field samples were tested. Since each sample was not large enough, samples of similar gradation were combined (Table D.5). The grain size distribution curves of the combined samples are presented in Fig. D.2c.

D.3 - RESULTS

The results for maximum and minimum densities obtained for the artificially graded TS samples, KS samples and TS field samples are presented in Tables D.3, D.4 and D.5, respectively. These tables present the basic grain size distribution parameters for each sample and the average maximum and minimum densities (minimum and maximum void ratios) obtained using the methodology described above. The use of the void ratios instead of densities has the advantage of normalizing the results in relation to the specific gravity of the grains (G). In the particular case of this study, all materials are quartz sands with G=2.65 and densities will be adopted for convenience. Each value of maximum or minimum densities presented on Tables D.3 to D.5 represent an average of 4 to 7 independent determinations. The coefficient of variation was on average 0.46% for the minimum density and 0.82% for the maximum density, indicating good reproducibility for these tests, since Tavenas et al.(1973) report a coefficient of reproducibility of 0.8% for each laboratory as an average for 40 laboratories across North America.

Coefficient of uniformity

Figure D.3 presents the variation of maximum and minimum densities with the coefficient of uniformity for artificially graded TS samples and for KS samples. It shows that both the maximum and the minimum densities increase as the coefficient of uniformity increases. This trend was expected since better graded materials (that have a larger C_U) have grains of various sizes and consequently there is a better chance of pores being filled with smaller grains, increasing the density of the sample. The same trend of increasing maximum and minimum densities with C_U was found by several other authors such as Burmister (1962), Youd (1973), Johnston (1973), Lacroix and Horn (1973), Poulos and Hed (1973) and Leary and Woodward (1973). Youd (1973) found a unique relationship between C_U and the reference densities (or void ratios) for 4 different sand mixtures. Youd also worked with artificially proportioned sand mixes with no fines. The results found here are consistent with Youd's results as shown in Figures. D.3b and D.3c. Youd's minimum void ratios are slightly low in relation to the values found here because he used a different method of determination of minimum void ratio (maximum density) that reportedly yields lower values. According to these plots, TS sand could be classified as subangular and KS sand as subrounded, which agrees with microscopic observations of these materials. Lacroix and Horn (1973) and Poulos and Hed (1973) found the relationship between the reference densities and the coefficient of uniformity to be relatively steep for low values of C_U but to level off for higher values of C_U ($C_U > 5$ or 6). The values of maximum density obtained in this study also compare well with the results presented by both Lacroix and Horn (1973) and Poulos and Hed (1973)

Figure D.4 gives the maximum and minimum densities for the TS field samples compared with the results obtained for the artificially graded samples. The values of maximum densities compare well, but the minimum densities of the field samples are generally low compared to the laboratory samples. This could be explained by the fact that the grains of the field samples have clay particles and salt deposits on their surfaces which increase the roughness of the surface, while laboratory samples have relatively clean grains. The rougher grains do not slide over other grains as easily, making a looser arrangement possible. The samples with more fines (Samples E, F and G) developed even lower minimum densities. An increase in the amount of fines is expected to increase the densities, and actually does (as discussed below), indicating that it is not the presence of fines itself that cause the lower minimum densities, but the increased roughness of the grain surfaces.

Mean particle size D_{50} and particle shape

The effect of D_{50} on the reference densities is presented on Figure D.5. For these samples, an increase in D_{50} caused an increase in both maximum and minimum densities. Burmister (1962), Kolbuszweski and Frederich (1963), Youd (1973), Dickin (1973) among others also found reference densities increasing as D_{50} increased (Figure D.5b). However, Youd (1973) did not get a unique relationship between density and D_{50} as he found when studying grain roundness. For this reason, Youd concluded that roundness is an important controlling factor and particle size by itself is not. Youd attributes the variation of reference densities with D_{50} to the fact that for most natural sands there is a correlation between particle size and particle shape. This is because natural processes tend to make larger particles more rounded. This conclusion agrees with the fact that Dickin (1973) found the effect of grain size being negligible for glass ballotini but not for quartz sands. An image analysis study on micrographs of the tailings sand used in this study shows indeed a slight increase in grain roundness with grain size.

Burmister (1962), Kolbuszweski and Frederich (1963), Youd (1973), Dickin (1973) and Holubec and D'Appolonia (1973) found that the more angular the grains, the lower are both maximum and minimum densities. Youd (1973) obtained unique curves for maximum and minimum void ratios versus roundness for several different sands (see Figure D.5c).

Figure D.6 shows that the TS field samples also displayed a trend of increasing reference densities with increasing grain size in a similar way as the artificially graded TS samples do. Again the maximum densities of field and laboratory samples compare well, but minimum densities of field samples are smaller than their laboratory counterparts.

Amount of fines (%F)

Figure D.7 presents the effects of the amount of fines on the reference densities for TS samples. Dry fines that passed the #200 sieve were collected and added in different proportions to the also dry sand to make up samples for this study. Due to this process, fines are not expected to adhere to the grain surfaces and contribute to increased grain roughness as it occurs in the field. The coefficient of uniformity was equal to 2.0 for all samples, but there was a small variation in D_{50} as indicated. Using a relationship between D_{50} and densities obtained from Figure D.5, the values of minimum and maximum densities presented in Figure D.7 were corrected to the value they would have if D_{50} were 0.150 mm and are presented in Figure D.8. This figure shows that, as expected, the reference densities increase with the amount of fines, but the rate of density increase is small, especially for the minimum density.

The reference densities of field TS samples versus amount of fines are presented on Figure D.9. In this figure the lower minimum densities of the field samples, especially the ones containing more fines, is very obvious.

Lacroix and Horn (1973) also found an increase in the reference densities as the percentage of fines increased from 0 to 7%. In this case also, the effect of the amount of fines was more pronounced on the maximum density than on the minimum density. Townsend (1973) varied the amount of fines between 0 and 25% and found that the reference densities increased first, but started decreasing after 15 to 20% fines for some samples.

D.4 - COMMENTS

The maximum and minimum densities of the TS field samples has already been determined by other workers. Figures D.10 and D.11 present a comparison between these results and the values obtained in this program. Except for the maximum densities measured by Law (1991) and Thurber (1985), all the other values seem to compare well with the values determined for this study. The scatter of the values seems similar for all studies. This scatter can be attributed to:

- coefficient of reproducibility normally associated with the method of determination of reference densities.
- statistical errors inherent in the method of determination of grain size distribution parameters.
- small differences in the shape of the grain size distribution curves from sample to sample, especially for the tails of the distribution. Samples with the same C_U and D_{50} can present different amount and size of fines below D_{10}. The situation is even worse for the fraction coarser than D_{60}. The Soviet use the parameter D_{90}/D_{10} which improves the control over the shape of the grain size distribution curve, although it does not solve the problem entirely. Burmister (1962) stresses the importance of the shape of the grain size distribution curve on the physical response of granular materials.
- presence of fines that adhere to the grains surface, possibly heterogeneously and in different degrees.

An indication that the difference in shape of the grain size distribution curves and the presence of fines on the grain surface may have an effect on the scatter of the field results is the fact that the tests on laboratory samples that had straight grain size distribution curves (Figures D.1 and D.2) and no fines on the grain surface showed little scatter (Figures D.3 and D.5). The same comment is valid for results found in the literature. Youd (1973) used artificially graded sand mixes prepared in the laboratory and obtained results with minimal scatter.(Figures D.3b, D.5b and D.5c, for example). Many other authors presented plots with considerable scatter when reporting results of natural (field) samples (see Figures D.3d, D.3e and D.3f).

The scatter of the field data causes variability in both ρ_{max} and ρ_{min} which corresponds to a significant fraction of the total range of densities ($\rho_{max} - \rho_{min}$). Therefore, the relative density varies over a wide range, depending on which ρ_{max} and ρ_{min} are assumed. For example, an in-situ density of 1.5 g/cm^3 would correspond to a relative density of 65% if lower bound values were adopted for the reference densities and only 25% if the upper bound values were chosen. This variability is so large that it would make the relative density meaningless in this case. However, such variability is comparable to the variability found in other studies reported in the literature. The coefficient of variation for these results (26 points) is 1.5% for the minimum density and 2.9% for the maximum density (or 1.1% without considering the data by Law, 1990 and Thurber, 1985). It is important to note that in this case all laboratories used different samples. Tavenas et al.(1973) report a study comparing reference densities determined by 40 different laboratories using identical samples, where the same level of variability was obtained when comparing the results from all laboratories. They concluded that "the quality of these results [ρ_{max} and ρ_{min}] would seem satisfactory with coefficients of variation of the order of 2.5% ..." (Tavenas et al.,1973, p.50). Tiedmann (1973) also reports similar coefficients of variability when comparing reference densities measured in 14 U.S. Bureau of Reclamation soil laboratories.

D.5 - REFERENCES

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TABLE D.1 - Range of variation of grain size parameters for the materials used for the study

Sand	Area	Range of CU	aver.CU	Range of %fines	aver.%F	Range of D50	aver.D50
TS	field test 1	1.49-3.10	1.85	1.6-11.6	4.8	0.157-0.213	0.183
TS	field test 2	1.51-3.30	2.14	2.6-15.4	7.1	0.135-0.196	0.170
TS	field test 4	1.73-2.93	2.40	4.9-19.3	12.4	0.114-0.168	0.137
			aver=2.1				aver=0.163
TS	lab. flume	1.39-3.15	1.65	0.2-4.9	1.40	0.120-0.286	0.178
KS	lab. flume	1.41-2.81	1.89	≈ 0	≈ 0	0.188-0.810	0.464

Table D.2 - TS sand artificially graded for the e_{max}/e_{min} study

SIEVE #	SOIL 1	SOIL 2	SOIL 3	SOIL 4	SOIL 5	SOIL 6	SOIL 7	SOIL 8	SOIL 9
40	0	0	0	0	0	8.5%	0	0	0
50	0	0	0	0	16%	15.5%	0	0	0
60	0	0	23%	14%	18%	13.5%	7%	1.5%	0
70	0	6%	18.5%	10.5%	11%	8%	12%	12.5%	9%
80	0	21.5%	21.5%	12.5%	12.5%	9.5%	12%	12%	11.5%
100	11.5%	22.5%	23%	13%	13%	10%	13%	12.5%	13.5%
120	23%	23%	14%	13%	13.5%	10%	13.5%	13.5%	13%
140	19.5%	20%	0	11%	11%	9%	12%	12%	12%
170	46%	7%*	0	13%*	5%*	10%*	13%	13%	13%
200	0	0	0	13%	0	6%	12.5%	13%	13%
passing 200	0	0	0	0	0	0	5%	10%	15%

NOTE.: * may include some material retained on the sieve #200

TABLE D.3 - Study of the variation of the reference densities for artificially graded TS sand

SOIL #	D_{50} (mm)	CU	% Fines	γ_{min} (g/cm^3)	γ_{max} (g/cm^3)	e_{max}	e_{min}	$\gamma_{max}/\gamma_{min}$	F* = $\Delta e/e_{min}$
1	0.11	1.5	0	1.349	1.654	0.964	0.602	1.23	0.601
2	0.15	1.5	0	1.381	1.686	0.919	0.572	1.22	0.607
3	0.20	1.5	0	1.423	1.700	0.862	0.559	1.19	0.542
4	0.15	2.0	0	1.427	1.703	0.857	0.560	1.19	0.530
5	0.20	2.0	0	1.462	1.729	0.813	0.533	1.18	0.525
6	0.20	2.5	0	1.499	1.763	0.768	0.503	1.18	0.527
7	0.138	2.0	5	1.426	1.719	0.858	0.542	1.21	0.583
8	0.128	2.0	10	1.411	1.712	0.878	0.548	1.21	0.602
9	0.120	2.0	15	1.413	1.715	0.875	0.545	1.21	0.606
OVERALL AVERAGE VALUES				1.421	1.709	0.866	0.552	1.20	0.569

* see Terzaghi (1925)

TABLE D.4 - Study of the variation of maximum and minimum densities for samples of KS sand

SOIL #	D_{50} (mm)	CU	% Fines	γ_{min} (g/cm^3)	γ_{max} (g/cm^3)	e_{max}	e_{min}	$\gamma_{max}/\gamma_{min}$	$F=\Delta e/e_{min}$
1	0.370	2.1	0	1.547	1.803	0.713	0.470	1.17	0.517
3	0.457	2.1	0	1.570	1.827	0.688	0.450	1.16	0.529
5*	0.414	3.2	0	1.671	1.938	0.586	0.367	1.16	0.597
2	0.400	1.8	0	1.546	1.773	0.714	0.495	1.15	0.442
4	0.578	1.9	0	1.559	1.782	0.700	0.487	1.14	0.437
AVERAGE VALUES		2.0	0	1.555	1.796	0.704	0.476	1.16	0.481

* not considered for the calculation of the averages because is out of the range of CU normally occuring for this sand (see Table D.1)

Table D.5 - Determination of Maximum and Minimum Densities for TS Field Samples

Soil	Field Samples	CU	% F	D_{50} (mm)	γ_{min} (g/cm^3)	γ_{max} (g/cm^3)	e_{max}	e_{min}	$\gamma_{max}/\gamma_{min}$	F
A	T1SG34/T2SG23/T1SG19-2	1.91	5.0	0.165	1.391	1.731	0.905	0.531	1.24	0.704
B	T1SG16/T1SG26	1.79	3.3	0.179	1.384	1.699	0.915	0.560	1.23	0.634
C	T2SG16-2/T2SG6/T2SG15	1.92	4.8	0.165	1.399	1.730	0.894	0.532	1.24	0.680
D	T1SG5/T1SG3-2	1.95	4.4	0.186	1.391	1.729	0.905	0.533	1.24	0.698
E	T1SG7/T1SG35/T2SG20	2.01	5.8	0.169	1.362	1.742	0.946	0.521	1.28	0.816
F	T2SG2/T4SG14/T2SG3	1.92	6.0	0.163	1.349	1.698	0.964	0.561	1.26	0.718
G	T4SG11/T4SG24	2.13	9.4	0.147	1.349	1.719	0.964	0.542	1.27	0.779
AVERAGE VALUES		1.95	5.5	0.168	1.375	1.721	0.928	0.540	1.25	0.718

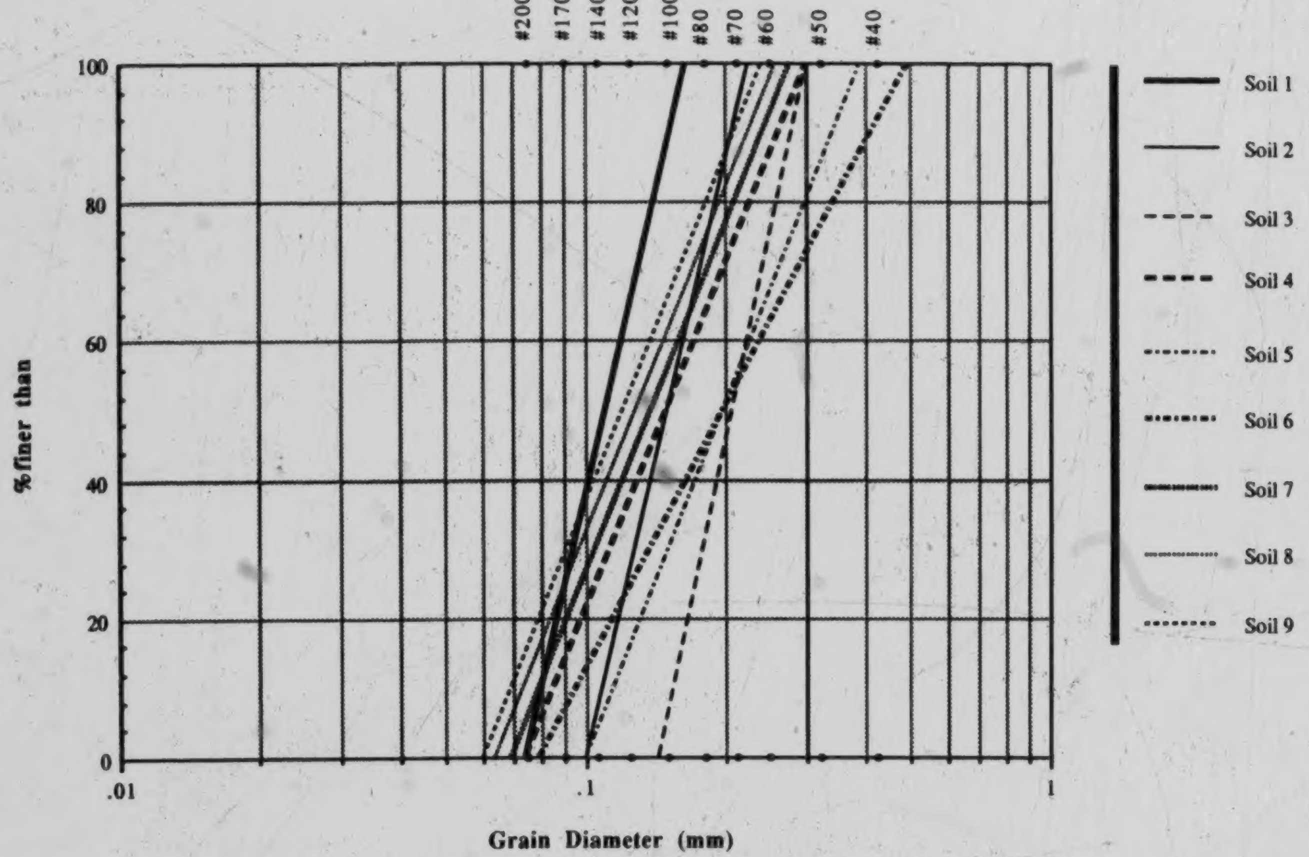

FIG. D.1 - Grain size distribution of the sieved TS soils used for the relative density study

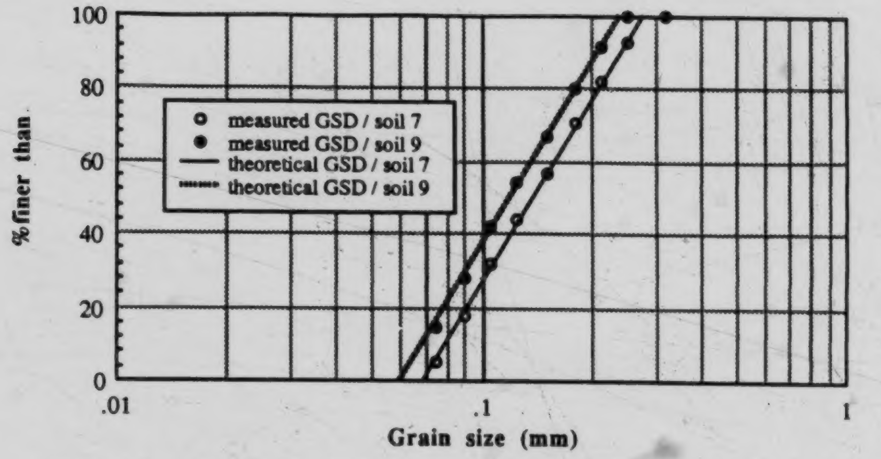

FIG. D.2a - Examples of actual grain size distribution for sieved TS soils

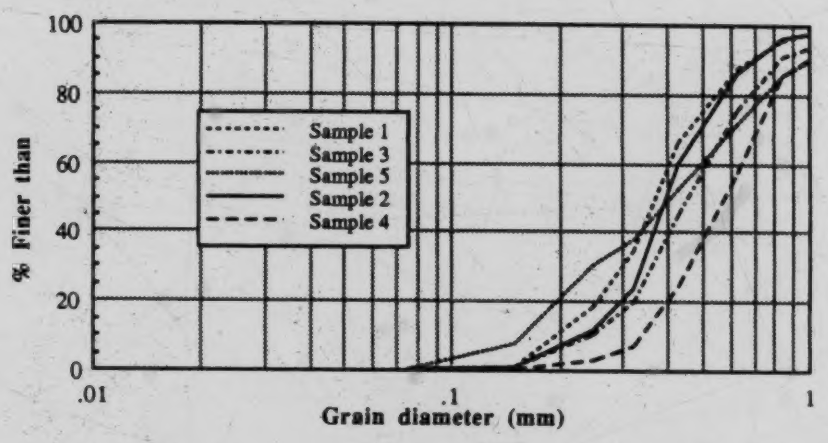

FIG. D.2b - Grain size distribution of KS samples

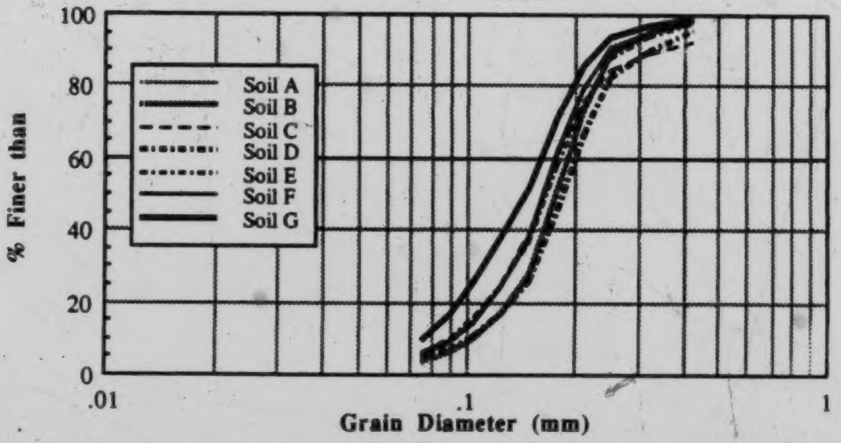

FIG. D.2c - Grain size distribution of the combined field samples (TS)

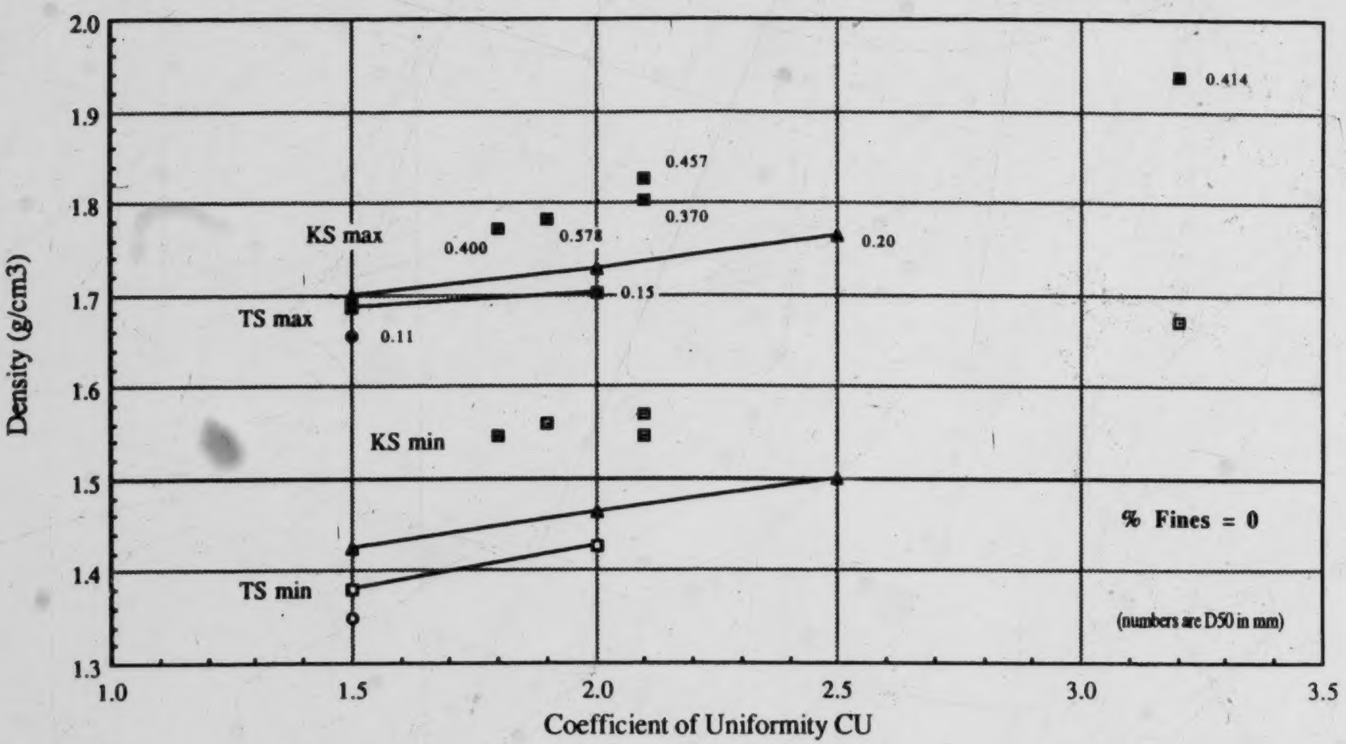

FIG. D.3a - Effect of CU and D50 on maximum and minimum densities (no fines)

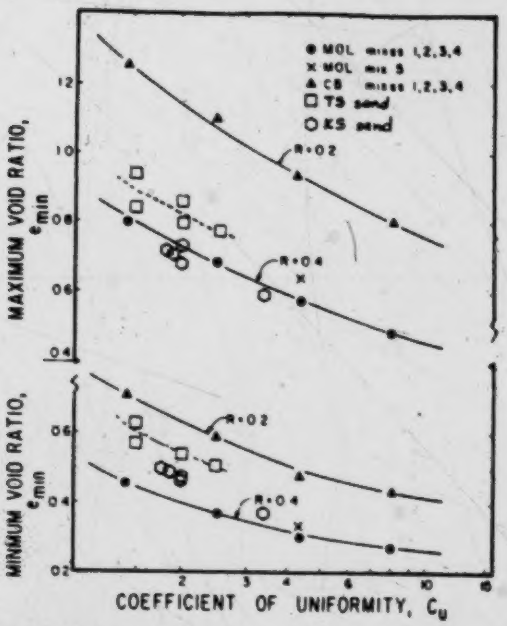

FIG. D.3b - Efect of CU and grain shape on maximum and minimum densities (modified after Youd, 1973)

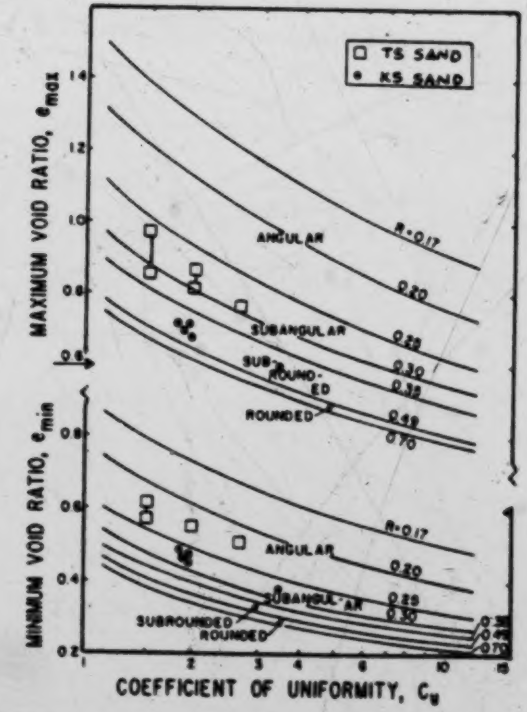

FIG. D.3b - Efect of CU and grain shape on maximum and minimum densities (modified after Youd, 1973)

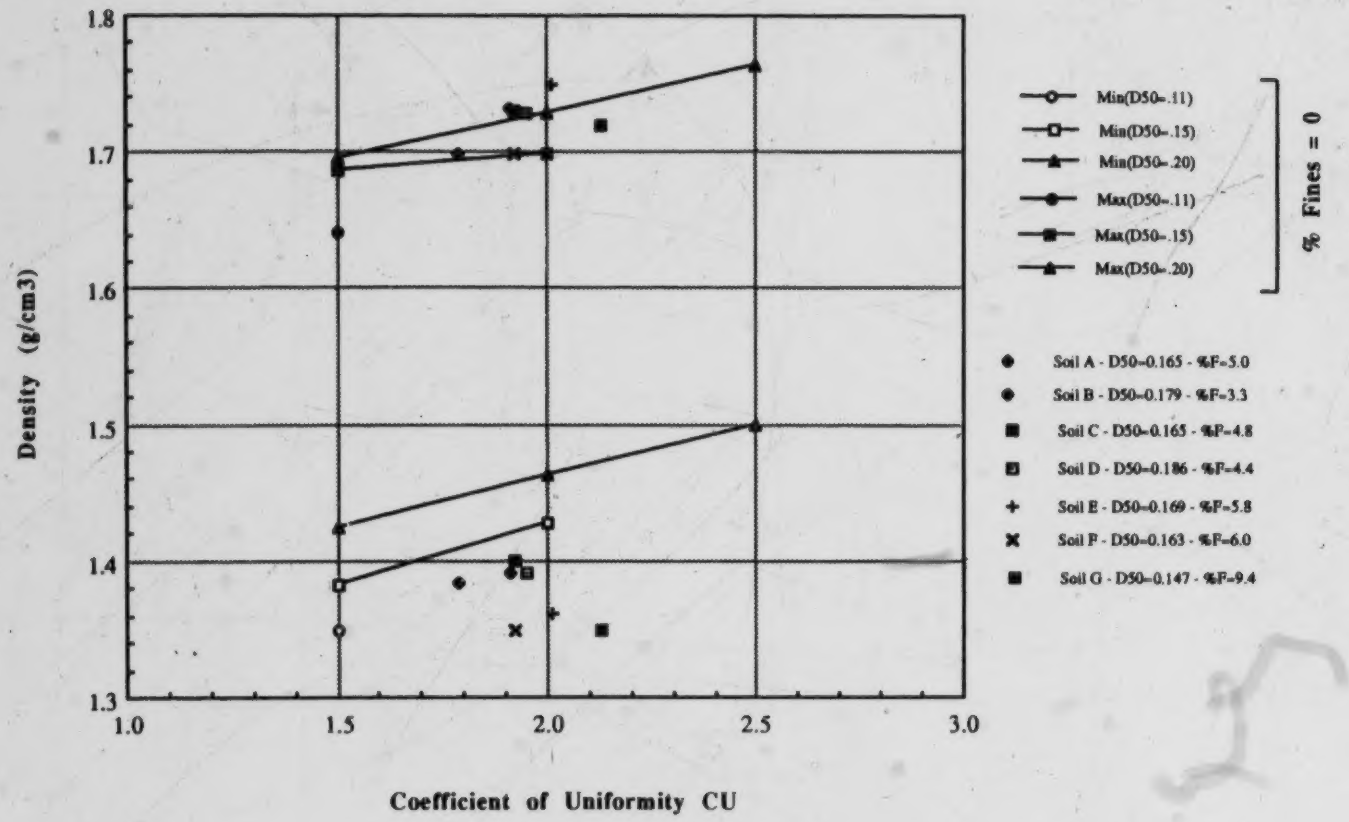

FIG. D.4 - Effect of CU and D50 on maximum and minimum densities of TS sand

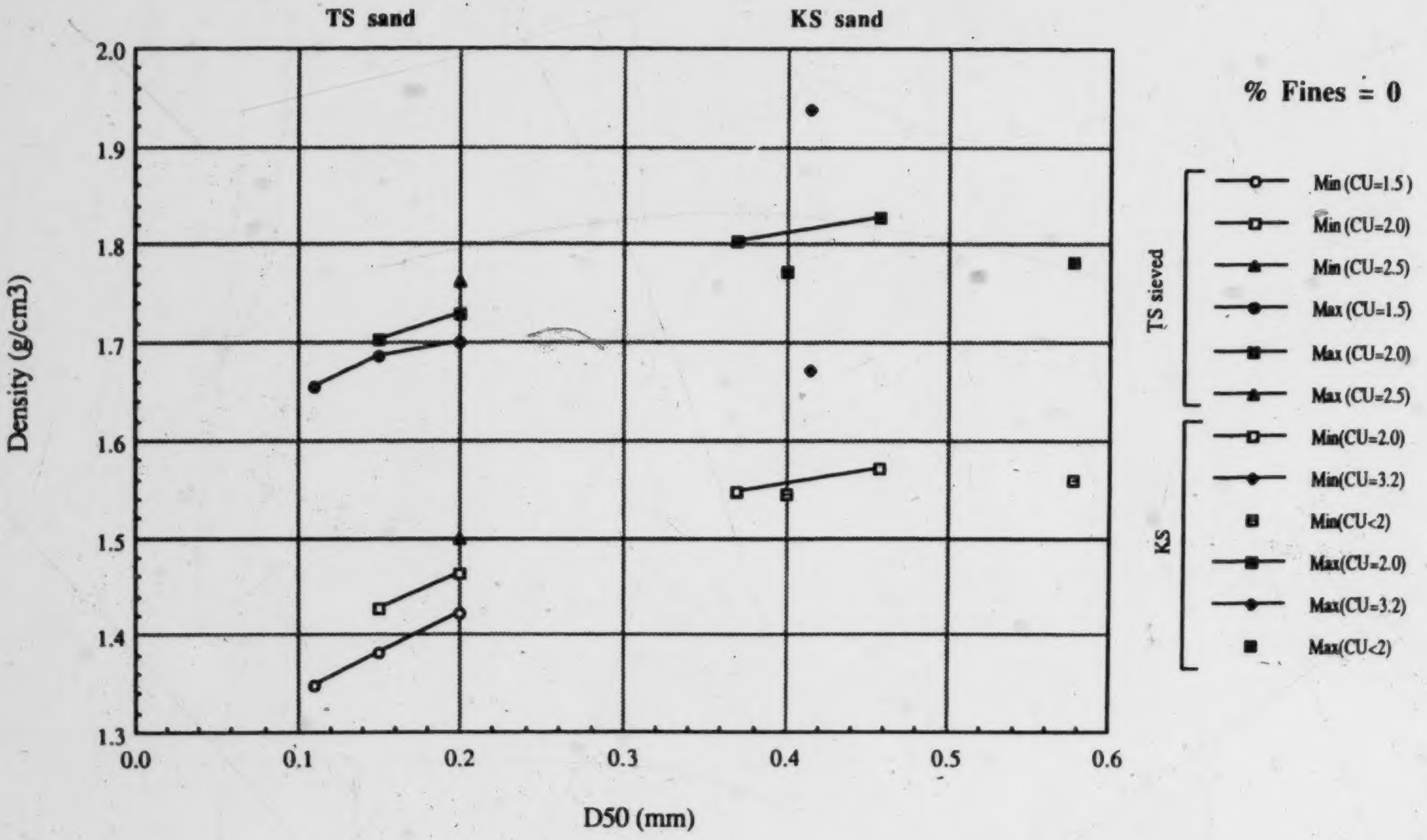

FIG. D.5 - Effect of D50 and CU on maximum and minimum densities (no fines)

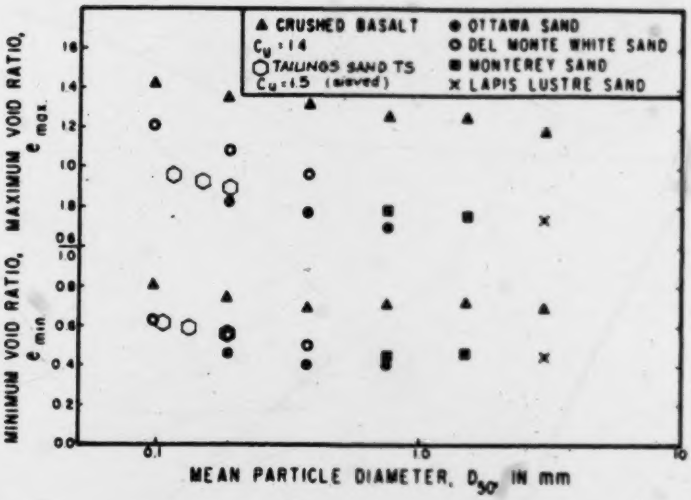

FIG. D.5b - Variation of maximum and minimum void ratios with grain size for different sands (modified after Youd, 1973)

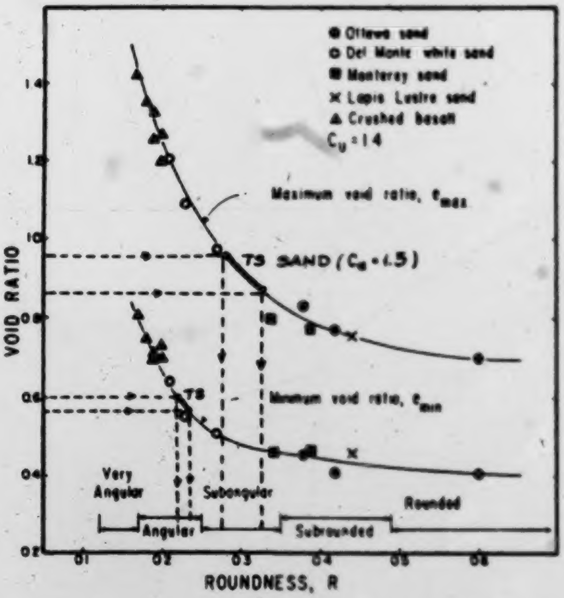

FIG. D.5b - Variation of maximum and minimum void ratios with grain roundness for different sands (modified after Youd, 1973)

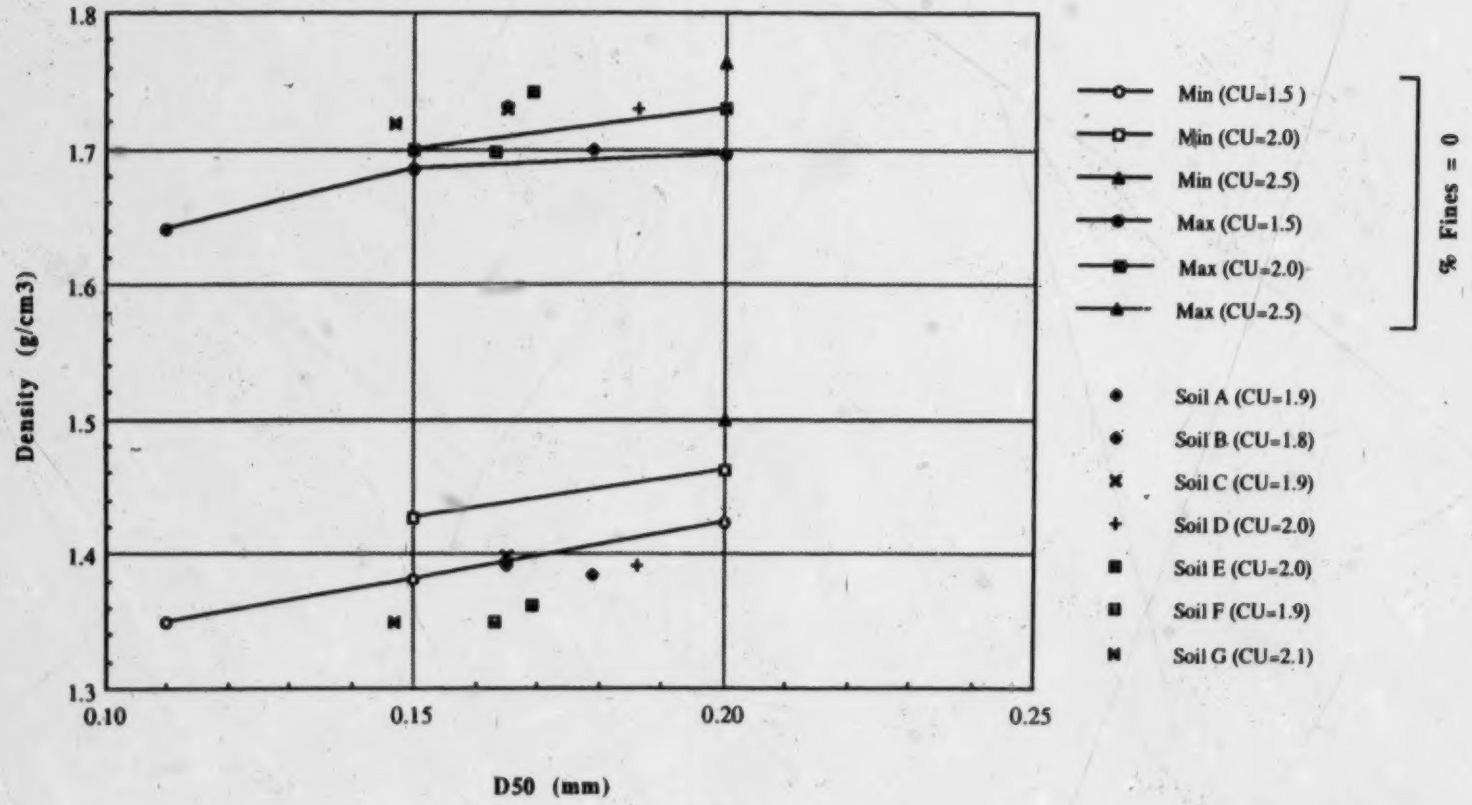

FIG. D.6 - Effect of D50 and CU on minimum and maximum densities of TS sand

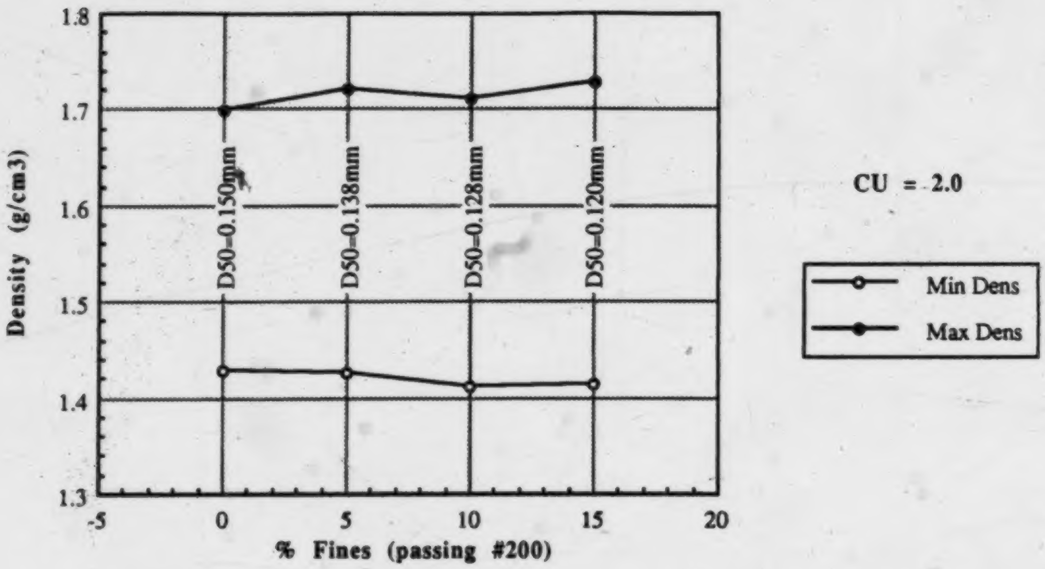

FIG. D.7 - Effect of the percentage of fines on the reference densities of the TS sieved samples

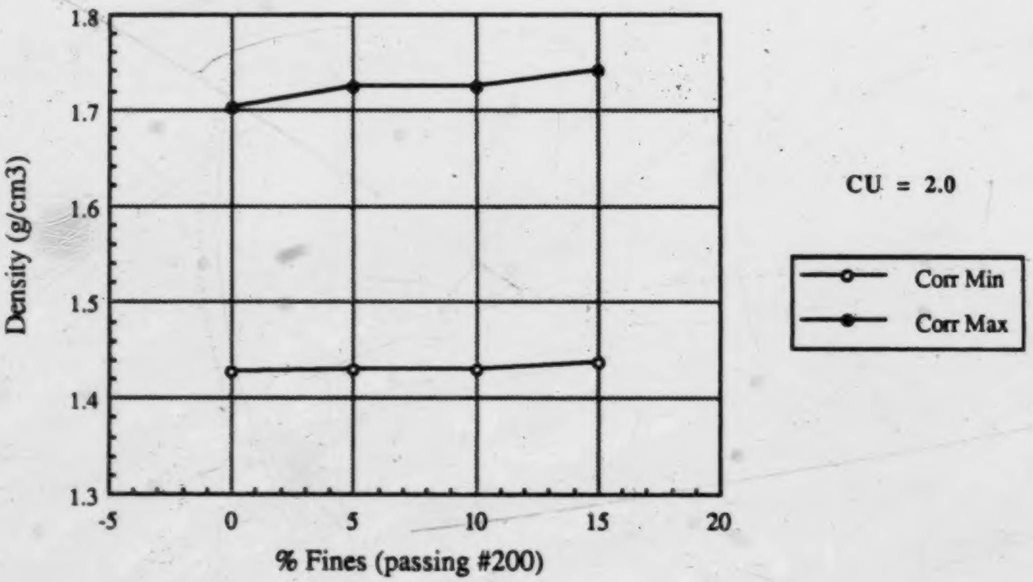

FIG. D8 - Amount of fines versus reference densities for TS samples, corrected to D50=0.15mm

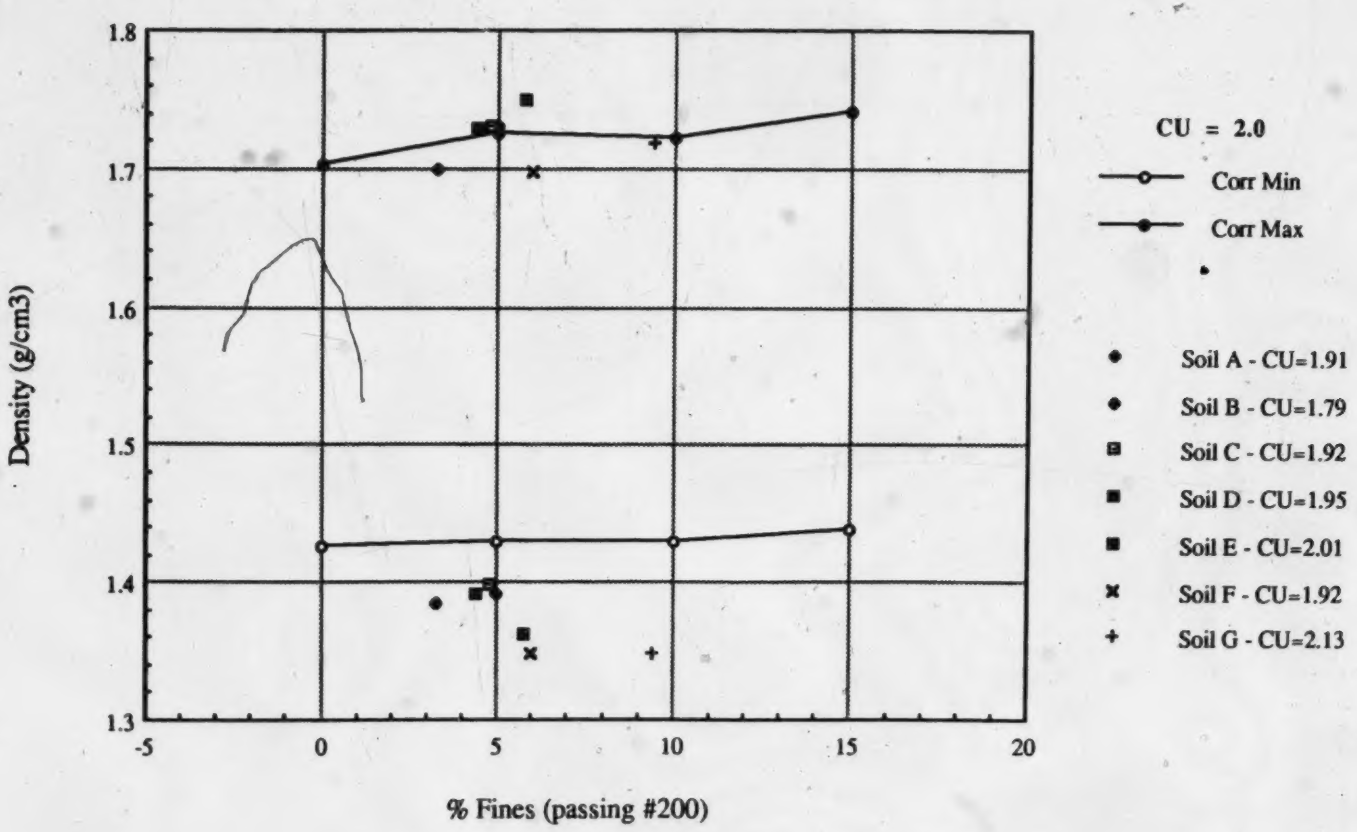

FIG. D.9 - Effect of amount of fines on maximum and minimum densities of TS sand

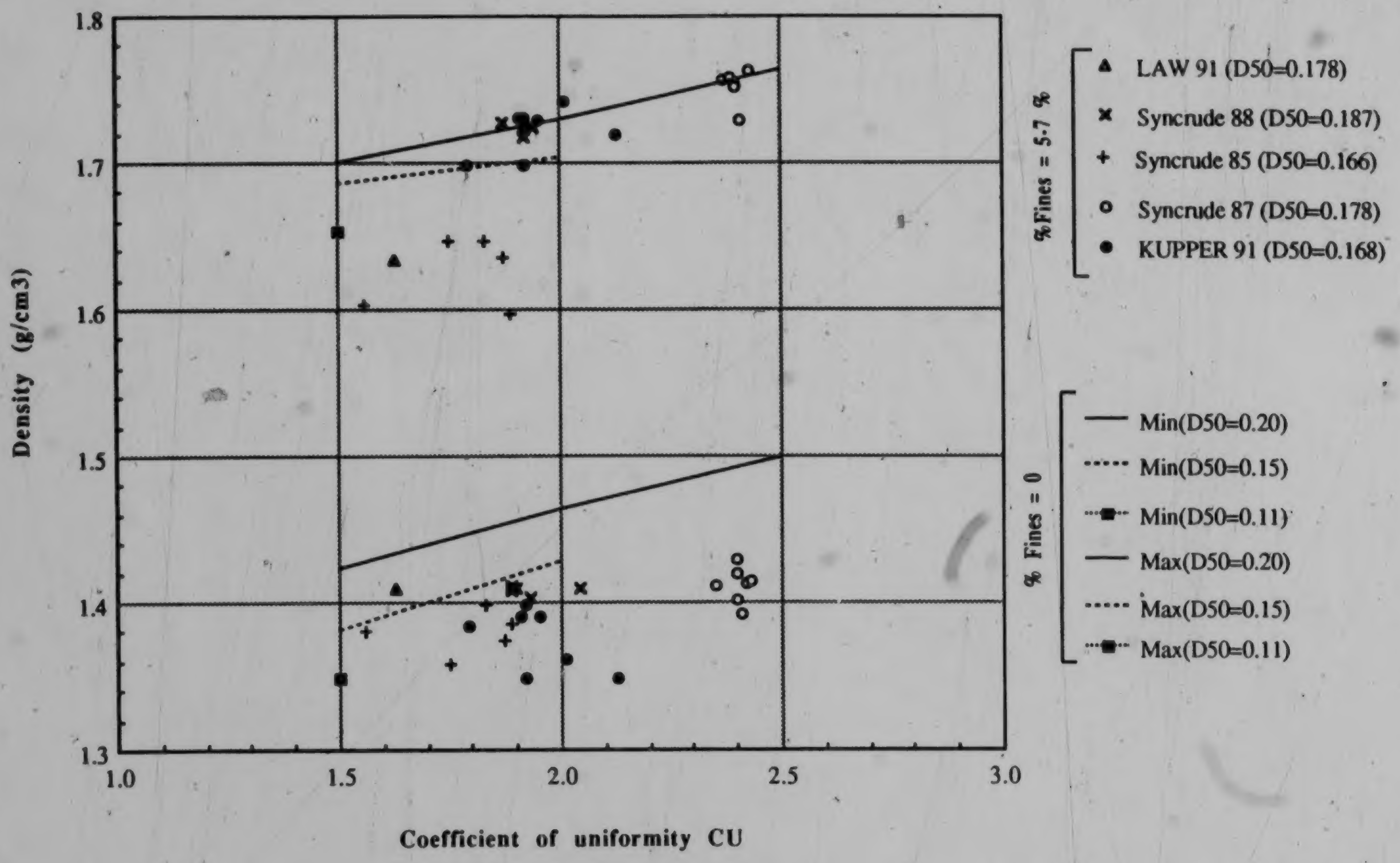

FIG. D.10 - Effect of the coefficient of uniformity on maximum and minimum densities of TS sand

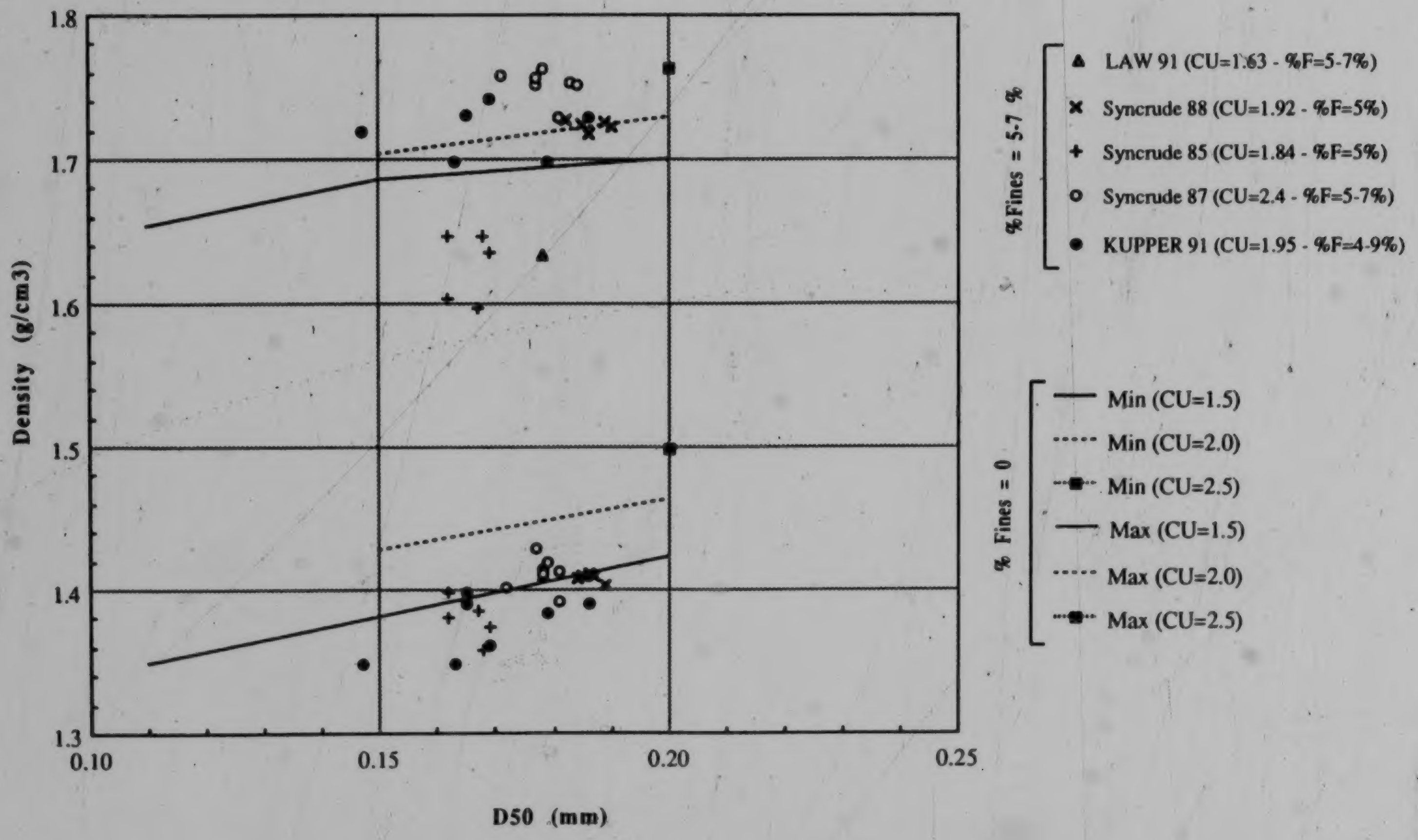

FIG. D.11 - Effect of D50 and CU on maximum and minimum densities of TS sand